

**ANALYSIS OF DEFORMATION BEHAVIOUR AT THE UNDERGROUND
CAVERN OF UPPER TRISHULI-1 HYDROPOWER PROJECT, NEPAL**

A Dissertation Submitted to the

CENTRAL DEPARTMENT OF GEOLOGY
TRIBHUVAN UNIVERSITY
Kirtipur, Kathmandu
Nepal

In Partial Fulfilment of the Requirement for the Award of Degree of Master of Science in
Engineering Geology.

By

MUKUNDA DHUNGANA
Year of Submission: 2023 AD
Exam Roll No.: E.G.E. 260/2075
Regd. No.:5237828-2012

RECOMMENDATION

This is to certify that **Mr. MUKUNDA DHUNGANA** has completed this dissertation work entitled “**ANALYSIS OF DEFORMATION BEHAVIOUR AT THE UNDERGROUND CAVERN OF UPPER TRISHULI-1 HYDROPOWER PROJECT, NEPAL**” as a partial fulfillment of the requirement of M.Sc. in Engineering Geology under my supervision. To my knowledge, this work has not been submitted for any other degree.

.....

Supervisor

Dr. Suman Panthee

Assistant Professor

Central Department of Geology

Tribhuvan University

Kirtipur. Kathmandu, Nepal

LETTER OF ACCEPTANCE

On the recommendation of the supervisor Dr. SUMAN PANTHEE, the dissertation work of **Mr. MUKUNDA DHUNGANA** entitled “**ANALYSIS OF DEFORMATION BEHAVIOUR AT THE UNDERGROUND CAVERN OF UPPER TRISHULI-1 HYDROPOWER PROJECT, NEPAL**” is accepted for the examination and is submitted to the board of examination for the partial fulfillment of the requirement of M.Sc. degree in Engineering Geology.

.....
Prof. Dr. Khum Narayan Paudyal
Head of the Department
Central Department of Geology
Tribhuvan University,
Kirtipur, Kathmandu Nepal

The dissertation work of **Mr. MUKUNDA DHUNGANA** entitled “**ANALYSIS OF DEFORMATION BEHAVIOUR AT THE UNDERGROUND CAVERN OF UPPER TRISHULI-1 HYDROPOWER PROJECT, NEPAL**” was examined by the following board of examiners and accepted for the partial fulfillment of the requirement of M.Sc. degree in engineering geology.

BOARD OF THE EXAMINE

Recommended by:

.....

Assistant Prof. Dr. Suman Panthee
(Supervisor)

Examined by:

.....

(External Examiner)

Examined by

.....

(Internal Examiner)

Accepted by:

.....

Prof. Dr. Khum Narayan Paudyal
(Head of the Department)

Date:

ACKNOWLEDGEMENT

Words cannot express my gratitude to assistant professor Dr. Suman Panthee for his incredible supervision, consistent support, enthusiastic encouragement, useful critiques, invaluable patience, and feedback.

I am deeply indebted to Professor Dr. Khum Narayan Paudyal, the Head of the Department of Central Department of Geology of Tribhuvan University for providing all the possible facilities, and necessary field equipment available at the laboratory store and laboratory facilities.

I would like to extend my sincere thanks to the librarians of the Central Department of Geology and Central Library of Tribhuvan University for providing me with the necessary literature review materials and to all the staff of the Central Department of Geology for their kind support and cooperation.

Furthermore, I would like to thank Upper Trishuli-1 hydroelectric Project members, my colleagues, and friends for their kind cooperation and support throughout my thesis work. Special thanks are moreover indebted to my field and research mates Mr. Prakash Pokhrel and Mr. Udaya Raj Sodari and my workmates Mr. Suman Senden and Miss. Prakriti Panta and all our design team of Power China for their kind cooperation and support in field visits, valuable time, invaluable discussion, consistent help, and encouragement.

Lastly, I would be remiss in not mentioning my family members, especially my father, mother, and wife Mrs. Sumitra Pandey for their consistent encouragement and support to make this journey successful, all teachers and staff of the Central Department of Geology, my classmates, and for their love, support, and inspiration. Their belief in me has kept my spirits and motivation high during this process.

.....

Mukunda Dhungana

ABSTRACT

The excavation of large caverns at lesser Himalayan rocks especially weak and schistose rock mass will be more prone to deformation and stability issues. The excavation of a large underground structure changes the initial stress regime forming tangential stress around the excavation surface. The redistribution of stress can result in localized stress concentration or areas of high stress may cause rock deformation or failure. This study presents the stress and deformation analysis of the powerhouse cavern of Upper Trishuli-1 using the numerical model and monitoring data from the extensometer.

In order to meet the objective, the analysis is conducted in three phases. At first, the input parameters are studied and analysed from the desk study and field observation. The input data are the geological and geotechnical data including the rock mass classification, discontinuity analysis, strength parameters, etc. Next, the cavern is modelled using numerical simulations (Phase²) to predict the deformation and stress behaviour. Finally, the predicted behaviour is compared with the monitoring data obtained from multipoint extensometers installed in the cavern.

The study shows that the distribution of major and minor principal stress was minimum at the excavation boundary and increases with depth. The deformation was maximum at the crown during the first layer excavation and after the final layer excavation, the deformation is maximum at side walls. The deformation data from the multipoint extensometer shows the absolute deformation is higher in between the depth of 5 and 10m anchor. The results shows that the deformation of rock mass is dependent to the geological condition and structures present in the excavated area. The comparison with the extensometer data shows that the measured behaviour is consistent with the predicted behaviour until the geological structures are present on the ground plays some crucial role on change in behaviour of rock mass. The study provides valuable insights into the performance of the powerhouse cavern and demonstrates the usefulness of extensometers in monitoring the deformation and stress behaviour of underground structures.

Keywords: Schistose rock, cavern, stress behaviour, deformation, geological structures, finite element method (FEM)

CHAPTER I.....	1
INTRODUCTION	1
1.1 Background.....	1
1.2 Objectives	2
1.3 Location and Accessibility.....	3
1.4 Topography and Drainage.....	3
1.5 Climate and Vegetation.....	4
1.6 Limitation	5
CHAPTER II	6
LITERATURE REVIEW	6
2.1 Literature review related to the geology of the Study Area.....	6
2.2 Literature review related to geotechnical properties of rock mass	7
2.3 Literature review related to stress and deformation analysis of cavern structure	9
CHAPTER III.....	13
MATERIALS AND METHOD	13
3.1 Materials	13
3.1.1 Maps and Data	13
3.1.2 Equipment.....	13
3.1.3 Computer Software and Program	14
3.2 Method.....	14
3.2.1 Desk Study.....	15
3.2.2 Field Study.....	15
3.2.3 Laboratory Work.....	22
3.2.4 Data Compilation, Data Processing, Numerical Modelling, and Interpretation.	23
CHAPTER-IV	28
RESULTS	28

6.2 RECOMMENDATIONS	80
REFERENCES	81
APPENDICES	I

LIST OF TABLES

Table 1 Empirical formulas for the estimation of rock mass deformation modulus	21
Table 2 Description of major discontinuities of powerhouse cavern	33
Table 3 Summary of RMR along the underground powerhouse cavern (UT1)	38
Table 4 Summary of GSI along the underground powerhouse cavern (UT1).....	38
Table 5 Summary of Q-Index along the underground powerhouse cavern (UT1).....	39
Table 6 Distribution of RMR, GS and RMR from Q system	40
Table 7 Calculation of uniaxial compressive strength of rock material	41
Table 8 The rock strength parameters along the powerhouse cavern are presented below .	41
Table 9 Estimation of in-situ deformation modulus (E_m) from RMR, Q, and GSI	42
Table 10 Hydraulic fracturing tests result, Project data.....	43
Table 11 Mechanical parameters adopted for numerical analysis of powerhouse caverns .	43
Table 12 Total displacement of Ex1 after first layer excavation	54
Table 13 Total displacement measured from extensometer and predicted from the model after first layer excavation	65
Table 14 Total displacement measured from extensometer and predicted from model after first layer excavation	69
Table 15 Total displacement measured from extensometer and predicted from the model after first layer excavation	70
Table 16 Relative displacement curve from the extensometer and numerical model	75
Table 17 Comparison of predicted and observed total displacement of Powerhouse cavern of Upper Trishuli-1 after first layer excavation from the model and multipoint extensometer (Ex1).....	76
Table 18 Total displacement after final layer excavation from the model at Ex5	76

LIST OF FUGURES

Figure 1 Location map of the study area	3
Figure 2 Drainage map of study area.....	4
Figure 3 Factors influencing tunnel stability (Panthi 2006)	16
Figure 4 Post Failure Characteristics of rock based on quality (Hoek and Brown, 1997) ..	21
Figure 5 Mesh generated with 3 nodded triangles.....	25
Figure 6 Computation of model using Finite element analysis	27
Figure 7 Geological map of study area.....	29
Figure 8 As-built geological map of Powerhouse caver.....	34
Figure 9 Stereographic Projection of discontinuities measured form Powerhouse cavern .	35
Figure 10 Probe-hole log of Powerhouse Cavern crown.....	36
Figure 11 Distribution of RMR, GSI, and RMR from Q along the powerhouse cavern.....	40
Figure 12 Distribution of sigma 1 stress magnitude at chainage 0+39m	45
Figure 13 Distribution of sigma 1 stress magnitude at chainage 0+39m	45
Figure 14 Distribution of sigma 3 stress magnitude at chainage 0+39m	46
Figure 15 Distribution of sigma 3 stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation.	46
Figure 16 Distribution of mean stress magnitude at chainage 0+39m	47
Figure 17 Distribution of mean stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation	47
Figure 18 Displacement of monitoring points after first layer excavation.....	48
Figure 19 Displacement of monitoring points after second layer excavation	48
Figure 20 Displacement of monitoring points after third layer excavation.....	49
Figure 21 Displacement of monitoring points after fourth layer excavation	49
Figure 22 Displacement of monitoring points after fifth layer excavation	50
Figure 23 Displacement of monitoring points after final excavation.....	50
Figure 24 Total displacement on extensometer1 (Ex1)	51
Figure 25 Total displacement on extensometer2 (Ex2)	52
Figure 26 Total displacement on extensometer3 (Ex3)	52
Figure 27 Total displacement on extensometer4 (Ex4)	53
Figure 28 Total displacement on extensometer5 (Ex5)	53

Figure 29 Total displacement measured from the extensometer at Ch 0+39	55
Figure 30 Distribution of sigma 1 stress magnitude at chainage 0+67m before excavation, first layer, and final layer excavation	56
Figure 31 Distribution of sigma 1 stress magnitude at chainage 0+67 m	57
Figure 32 Distribution of sigma 3 stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation	57
Figure 33 Distribution of sigma 3 stress magnitude at chainage 0+67 m	58
Figure 34 Distribution of mean stress magnitude at chainage 0+67m before excavation, first layer, and final layer excavation	58
Figure 35 Distribution of mean stress magnitude at chainage 0+67 m	59
Figure 36 Displacement of monitoring points Ex1 after first layer excavation	59
Figure 37 Displacement of monitoring points Ex1 after second layer excavation.....	60
Figure 38 Displacement of monitoring points after third layer excavation.....	60
Figure 39 Displacement of monitoring points after fourth layer excavation	61
Figure 40 Displacement of monitoring points after fifth layer excavation	61
Figure 41 Displacement of monitoring points after final excavation.....	62
Figure 42 Total displacement on extensometer1 (Ex1)	63
Figure 43 Total displacement on extensometer2 (Ex2)	63
Figure 44 Total displacement on extensometer3 (Ex3)	64
Figure 45 Total displacement on extensometer4 (Ex4)	64
Figure 46 Total displacement on extensometer5 (Ex5)	65
Figure 47 Deformation measurement of the powerhouse cavern roof from Ex1 at Ch 0+67m.....	67
Figure 48 Total displacement curve of measured and predicted data (Ch 0+39 m) after 1st layer excavation.....	69
Figure 49 Total displacement curve of measured and predicted data (Ch 0+67 m) after 1st layer excavation.....	70

LIST OF PHOTOGRAPHS

Photograph 1 Field equipment used for the study	14
Photograph 2 Uniaxial compressive strength test in laboratory	23
Photograph 3 Interbedding of the psammitic schist and pelitic schist observed in the Upper Trishuli-1 Powerhouse Cavern at the confluence of the Trishuli River and Mailung Khola	30
Photograph 4 Interbedding of the wavy phyllite with quartzite rocks observed 500m south of the Tiru valley.....	30
Photograph 5 Thick to a massive bed of the Ulleri Augen Gneiss observed at about 1 km from Mailung Bajar along the traverse of Mailung to Tiru village	31
Photograph 6 The geological contact of the Kuncha Formation and the Ulleri Augen Gneiss observed at about 500 m north of the Powerhouse of the project Upper Trishuli-1.....	31
Photograph 7 Distribution of schist unit at powerhouse cavern.....	32
Photograph 8 Combination of potentially unstable structural surfaces on the downstream side wall of the powerhouse cavern.....	33

LIST OF ABBREVIATIONS

2D	Two Dimensional
3D	Three Dimensional
BEM	Boundary element method
D	Disturbance Factor
E_m	Deformation modulus
FEM	Finite Element Method
GIS	Geographic information system
GPS	Global positioning system
GSI	Geological Strength Index
HCL	Hydrochloric acid
IS	Indian Slandered
ISRM	International Society of Rock Mechanics
Ja	Joint alteration
Jn	Joint number
Jr	Joint roughness
Jv	Joint volume
m	meter
MCT	Main Central Thrust
mm	millimeter
MPa	Mega pascal
MR	Modulus ratio
NEA	Nepal Electrical Authority
Q	Rock quality index
RMR	Rock mass Rating
RQD	Rock Quality Designation
SCR	Surface Condition Rating
SR	Structure rating
SRF	Stress reduction factor
UT1	Upper Trishuli-1

CHAPTER I

INTRODUCTION

1.1 Background

Nepal is one of the most developing countries in the world with mountainous topography and huge water resources. Hydroelectricity generation is one of the major economic backbones of the country. The hydroelectric potentiality of generating in Nepal is about 83,000 MW (Shrestha 1998), out of which 42,000 MW is economically feasible. However, Nepal has successfully developed about 2190 MW so far (NEA 2022). This has been working on increasing its renewable energy usage lately, and the Upper Trishuli-1 Hydroelectric Power Project (UT1) is one of the major new contributors to this project is a runoff river scheme with 216 MW capacity electricity generation and is considered one of Nepal's largest runoff river schemes.

Constructing a powerhouse cavern in an active stress zone presents several engineering challenges due to the geological instability and deformation in the region. Deformation of caverns refers to any change in their shape or size caused by internal or external factors. There are two types of deformation: elastic and plastic. Elastic deformation is temporary and reversible, while plastic deformation is permanent due to rock mass yielding or failure. The deformation of caverns is influenced by various factors, including rock properties, type of load, duration of the load, and interaction with surroundings. The deformation modulus, strength, and jointing of the rock mass are crucial in determining the deformation behaviour of a cavern.

In designing a powerhouse cavern, stability analysis through numerical and analytical methods is crucial. Deformation analysis of tunnels involves measuring, predicting, and interpreting deformation patterns in and around the tunnel. In-situ monitoring, laboratory testing, analytical modelling, and numerical modelling are some of the methods used for deformation analysis. In-situ monitoring involves the installation of sensors and instruments to measure tunnel and ground deformation. For numerical modelling, mechanical properties of rock samples are determined from laboratory testing, and computer simulations can model complex geotechnical conditions and provide detailed information on deformation patterns

and causes. The deformation analysis was done by different researchers in different large caverns.

The Upper Trishuli-1 Hydroelectric project has an underground powerhouse cavern with dimensions of 80 m in length, 20.6 m in width, and 39.5 m in height with a buried depth of 300 to 320 m with its longitudinal axis of east to west. Three penstocks upstream, three IPB tunnels, and three draft tube tunnels downstream relate to the powerhouse. Similarly, a large transformer cavern of Length 27.75m, width 15.6m, and Height 12.3m is connected to the powerhouse cavern. Geologically, the study area belongs to the rock of pelitic and psammitic schist. For the excavation and stability analysis of the Cavern, numerous considerations need to be considered including the regional and local geology, and mechanical properties of the rock (elastic modulus, Poisson's ratio, Cohesion Friction angle, and unit weight). Crustal Stress (tectonic stress, gravitational stress, in situ stress, topographic stress, and residual stress). Rock mass properties (rock type, rock strength, rock quality designation, block size, the density of joints, joint properties such as Joint geometry, joint spacing, aperture, infilling material, roughness), Groundwater condition, weakness zone (sheared zone, weak intercalated layer) burial depth and geometry and location of the cavern.

In this research work, 2D numerical modelling using finite element method through Phase2 software was used to analyse the stability conditions of a powerhouse cavern of Upper Trishuli-1. The stress distribution and deformation analysis were studied, and the modelling result was compared with instrumented data from a multi-point extensometer. This approach provided valuable insights into the behaviour of the powerhouse cavern and helped ensure its stability and safety. The results of this research can be used to inform future designs of powerhouse caverns and improve the safety and longevity of such structures.

1.2 Objectives

The main objectives of the study are to analyse the deformation behaviour and stress distribution in different chainages of the powerhouse cavern of the Upper Trishuli-1 Hydroelectric Project. This involves evaluating the potential for ground movement, displacement, and deformation. The main objectives for the study are given below:

- To understand the geological characteristics of the Upper Trishuli-1 powerhouse area.
- To study geotechnical properties and evaluate the input parameters for stability analysis of the cavern.

- To apply numerical and in-situ monitoring methods for the assessment of cavern deformation.
- To correlate the predicated and monitored deformation of powerhouse cavern.

1.3 Location and Accessibility

The study area is situated in the northwest of Kathmandu, in Uttargaya rural municipality, ward no. 1, Mailung of Rashuwa district. Geographically, the study area location is situated between longitudes $85^{\circ}12'40''$ and $85^{\circ}18'03''$ E, and latitudes $28^{\circ}04'27.50''$ and $28^{\circ}7'42''$ N. It is 10km away towards North from Trishuli.

The study area is accessible from Kathmandu via the Pasang Lhamu Highway and Galchi - Mailung - Rasuwagadhi road with 80 km to the northwest of Kathmandu. Along with major highways, different feeder routes within the study area make it easy for transportation. The study area covers the part of the topographic map with sheet numbers 2885 13 (SOMDAN) prepared by the Survey Department, Government of Nepal in 1996 AD. The location of the study area is shown in Figure 1.

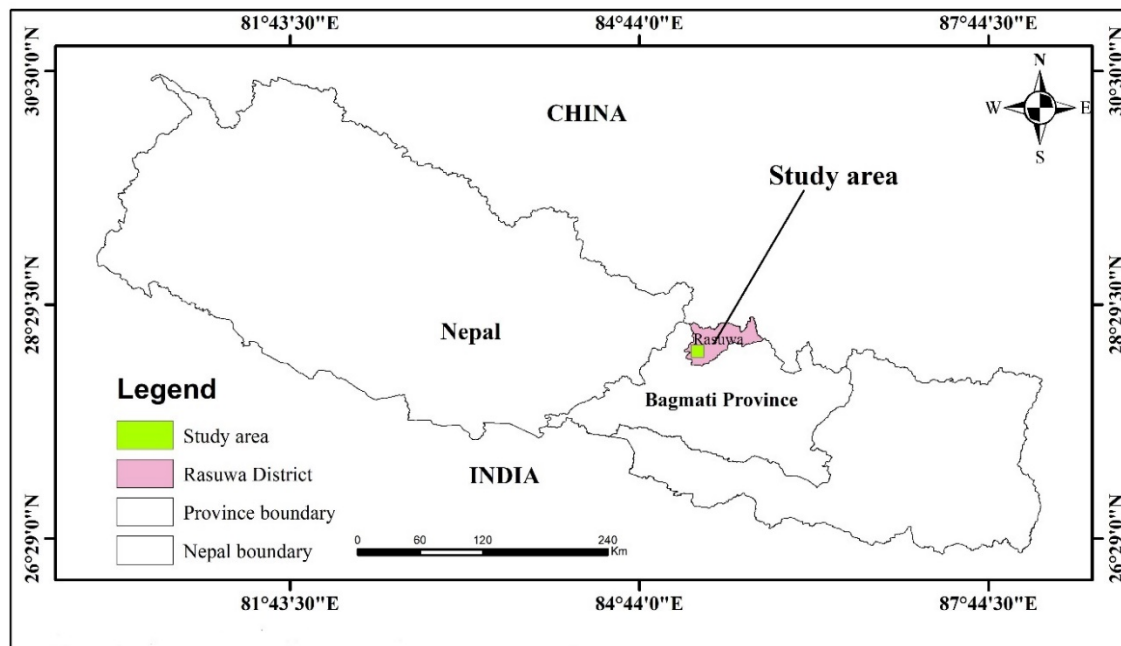


Figure 1 Location map of the study area

1.4 Topography and Drainage

Geologically, the study area lies in the Lesser Himalayan Zone. The area is characterized by matured topography with ridges and spurs either forming cliffs or steep faces. The change in

topography is one of the influencing factors for change in stress behaviour and deformation of cavern. The highest altitude is 1866 m at the south of Tiru village and the lowest one is 960 m at the confluence of the Mailun Khola and Trishuli River. The right bank of the river is steeper than the left Bank. The Trishuli is the main river of the study area, and its tributary are Mailun Khola and Nyam Nyam Khola. Many other gullies and small tributaries are connected to these rivers. The overall dendritic drainage pattern was observed in the study area. The drainage map of the project area is shown in Figure 2.

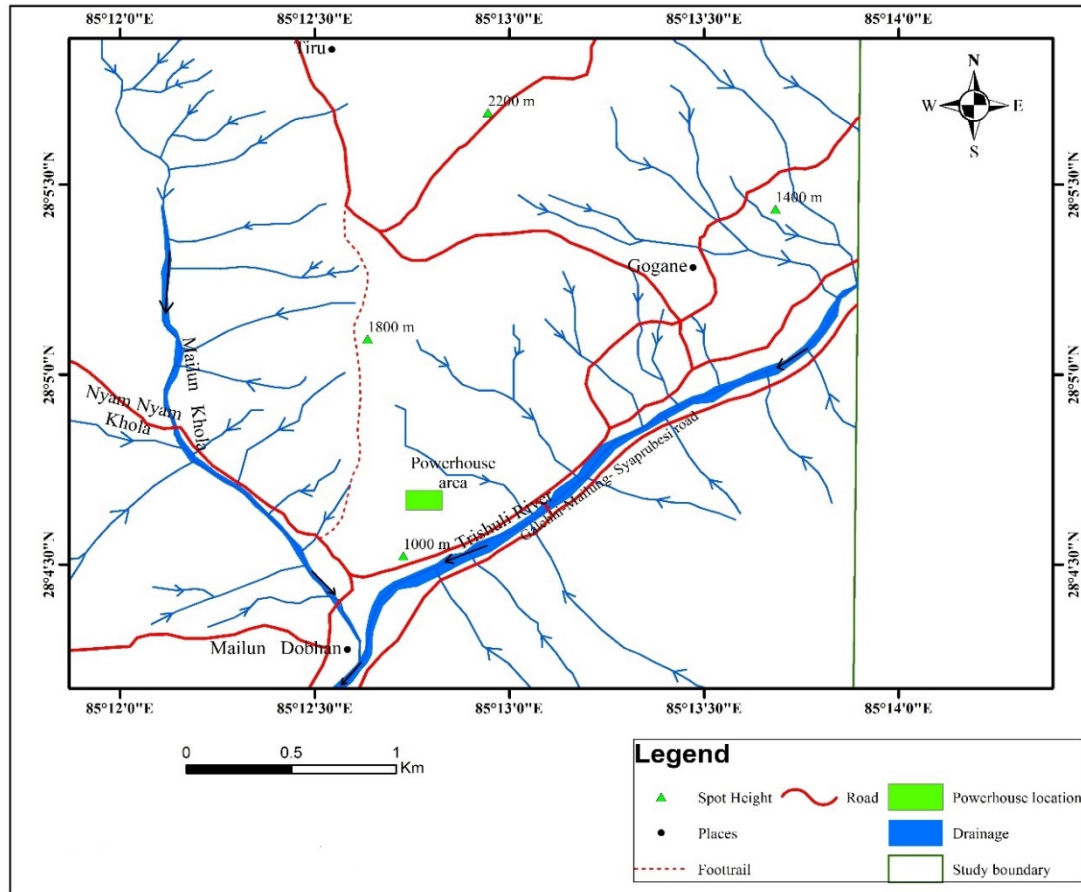


Figure 2 Drainage map of study area

1.5 Climate and Vegetation

The study consists of diverse topography and climatic conditions. The basin in the upper part is surrounded by steep hills and has a sub-tropical to tropical climate. In general, the amount of precipitation is highest to the south at the lower elevations and gradually decreases to the north with an increasing elevation. The average annual rainfall is 994.3 mm and occurs mainly during June, July, and August. While the average maximum temperature in Dhunche

is 22.6oC, the minimum temperature stands at 2oC (Department of Hydrology and Meteorology 2020). Changes in climate, such as increased precipitation or temperature, can cause water infiltration, leading to an increase in groundwater within the rock mass. This can lead to increased weathering of rock mass, which can change its properties and weakening and instability of the rock mass. This area is rich in flora due to variations in altitude, temperature, and rainfall, one can find different vegetation as moved from the area of one altitude to another. At low latitudes, the deciduous forest is found and at high altitudes, coniferous forest, trees, shrubs, herbs, and grassland are found. Because of the overburden depth of cavern is high, there will be no effect of flora on the cavern along the study area.

1.6 Limitation

- Numerical modelling was conducted to simulate the behaviour of the rock mass in the study area. However, certain components such as penstock tunnel, IPV tunnel, draft tube tunnel, and construction adit tunnels were not included in the modelling. This may have impacted the accuracy of the results obtained.
- In addition, the software used for numerical modelling had certain limitations, including assumptions and simplifications made in the analytical and numerical models. These simplifications may not fully represent the complexity of the actual rock mass behaviour, and may have resulted in inaccuracies in the results obtained.
- The laboratory and field test results used to inform the numerical modelling also had limitations. The limited number of samples used and the inability to capture the heterogeneity of the rock mass may have impacted the representativeness and accuracy of the results obtained.
- Finally, the construction of the powerhouse cavern is not yet complete, and the deformation data obtained from the multipoint extensometer may change in the future. This should be considered when interpreting the results obtained from the numerical modelling and other analyses.

CHAPTER II

LITERATURE REVIEW

Deformation analysis of underground structures is a major concern for researchers due to its potential impact on the safety and stability of the structure. Several factors can contribute to deformation, including the geology of the area, the properties of the rock mass surrounding the structure, and the stress conditions present. Given the complexity of deformation in underground structures, researchers from various fields have contributed to this area of study. Their work has focused on understanding the underlying mechanisms that contribute to deformation, as well as developing methods for accurately measuring and predicting deformation. To gain a comprehensive understanding of deformation, researchers have conducted field studies to gather data on the behaviour of underground structures in real-world conditions. They have also conducted laboratory experiments to investigate the properties of rock mass and to develop numerical models that can simulate the deformation of underground structures.

In addition to understanding the underlying mechanisms of deformation, researchers have also developed various methods for measuring and predicting deformation. These include monitoring systems that can detect changes in deformation over time, as well as analytical models that can predict deformation based on the properties of the surrounding rock mass.

The different researchers work in deformation along with concern field's study works were reviewed. The review work was carried out in following subheadings:

2.1 Literature review related to the geology of the Study Area

Stocklin and Bhattarai (1977; 1980) divided the rocks of Central Nepal into Nawakot complex and the Kathmandu complex separated by Mahabharat Thrust (MT) based on aerial photo interpretation and fieldwork. They correlated the MT with MCT in the north. According to this study, the study area falls on the rock zone of the Nawakot Complex.

K.C and Paudyal (2019) studied the rock succession exposed in Ramche-Chhyamthali area and the Ulleri augen gneiss have been mapped at the confluence of the Trishuli and Mailung Khola of the Rasuwa district surrounded by the Kuncha Formation.

Venkatachalam and Singh (2017) made the study of schist from Lesser Himalaya and showed that schist with a high content of mica and quartz had a lower deformation modulus and higher axial strain compared to schist with a high content of feldspar and hornblende. The presence of weak minerals like sericite and chlorite also resulted in a lower deformation modulus and higher axial strain. The paper concluded that the mineralogical composition plays a significant role in the deformation behaviour of schist under triaxial compression, and understanding this relationship can aid in the prediction of rock deformation in engineering projects.

2.2 Literature review related to geotechnical properties of rock mass

A study by Khandelwal et al. (2018) investigated the strength properties of schist from the Lesser Himalayas in India. The study found that the UCS of the schist ranged from 35 to 85 MPa, with an average value of 57 MPa. The researchers also found that the UCS of the schist was positively correlated with the quartz content and negatively correlated with the clay content of the rock.

Rock Mass Properties

Rock mass classification is a widely used method for assessing the behaviour of rock masses in engineering and design of projects. The review on rock classification basically involves analysing the studies that has been done previously on the subject, including the development of classification system and their application on engineering studies.

Terzaghi (1946) categories the rock mass quantitatively from rock class I to rock class IX, that includes the hard and intact rock to weak and swelling rock mass based on discontinuities present in rock mass. This method is purposed to evaluate the rock load that steel arches used to tunnel support must bear.

Deere et al. (1967) developed a quantitative assessment of rock mass quality using drill cores. if no core is available but discontinuity traces are visible in surface exposures or exploration audits, the Rock Quality Designation (RQD) is calculated from the number of discontinuities per unit volume, as explained by Palmström (1982).

A new system known as the Rock Mass Rating (RMR) system or Geomechanics classification was developed in South Africa (Bieniawski 1972; 1973) to estimate stability and support of tunnels. The RMR was revied in 1974, 1975, 1976, and 1989 provide the rock

mass classification for the determination of rock mass quality and support design and develop the evaluation based on six parameters: uniaxial compressive strength (UCS), rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions, and orientation of discontinuities.

Several modifications and variations of the RMR system have been proposed over the years to improve its accuracy and applicability to specific geological conditions. For mining purposes, Laubuscher (1977; 1984) further refined the RMR as Modified Rock Mass Rating (MRMR) system.

Based on 200 case studies, Barton et al. (1974) proposed the Tunnelling Quality Index (Q) methodology. The tunnel quality index(Q) value indicates how stable the rock mass is for an underground opening in a jointed mass. The six factors that the Q-system recommended for the classification of rock masses and the assessment of tunnel support are the Rock Quality Index (RQD), joint set number (Jn), joint roughness number (Jr), joint alteration number (Ja), joint water reduction factor (Jw), and stress reduction factor (SRF). The Q-System was later improved by Grimstad and Barton (1933) using data from 1000 case studies.

Hoek and Brown (1977) developed the Geological strength index (GSI) method for hard and weak rock masses based on visual observation of the rock exposure, as well as on structure and discontinuity surface condition. After using the knowledge obtained from applying GSI to real world engineering issues, various modifications were made. Hoek (1994) and Hoek et al. (1980; 1995; 1997; 2007). have published numerous books, articles, and textbooks on the topics of rock failure criteria, rock mechanics, and tunnelling. The GSI system is based on the rock mass quality index, which is obtained by evaluating the structure of the rock mass, the condition of the joints, and the condition of the groundwater.

The properties of rock masses are affected by the geological and structural features which includes the rock type, the orientation of discontinuity, and the grade of weathering and alterations of joints. The rock mass properties are estimated by laboratory experiment which includes the uniaxial compressive test, Point load test etc. Similarly, in-situ test as well as empirical approaches can be used to estimate the strength of rock mass. Different methods are developed to estimate these properties, such as lab experiments, field tests, and empirical approaches.

Deformability

Bieniasaski (1993), provided the empirical relationships for estimating the strength of rock mass which is based on simple equation model. Bieniawski (1978) purposed a relationship between the deformation modulus of rock for the rock mass having higher value than 50 RMR. Later, Serafim and Pereira (1983) provided the relationship for rock mass rating less than 50 RMR.

Deere et al. (1967), Gardner (1987) and Zhang and Einstein (2000), develop the correlation between rock quality designation (RQD) and Modulus Ratio (MR) E_m/E_r . Where E_m is deformation modulus of rock mass and E_r is Deformation Modulus of intact rock.

Hoek and Brown (1997) modified the relationship of Serafim and Pereira (1983). Hoek et al. (2002) give the empirical relationship for deformation modulus based on Geological Strength Index (GSI) and uniaxial compressive strength of intact rock material.

Barton (2002) purposed a relationship for deformation modulus based on Tunnel quality index (q) and uniaxial compressive strength of intact rock material.

More recently, Hoek and Diederichs (2006) derived the relationship between rock mass deformation modulus E_m , GSI, and Intact rock deformation (E_r) after large number of insitu measurement from China and Tiwan.

Panthee et al. (2016) made a comparative study along the tunnel alignment of the Kulekhani III Hydroelectric project using an empirical relation for estimating the deformation modulus. The study found that a large disparity in the E_m values was observed from different empirical equations, which is due to the sensitivity of the mathematical function to the rock parameters. The comparison shows the sensitivity in E_m value calculation is high on higher rock mass and on lower rock mass. Therefore, caution must be taken when using these equations in engineering design and analysis, and site-specific testing and calibration should be performed whenever possible.

2.3 Literature review related to stress and deformation analysis of cavern structure

One of the earliest studies on this topic is by Sapigni et al. (2003) characterize the engineering geological condition of Powerhouse cavern in Italian alps and made a comparison between the predicted and measured deformation which is based on two numerical model (FEM and DEM codes). The models were used to investigate the overall

stability of the cavern and analyse the expected deformation that occurs during excavation by phase2 and concluded that the measurement of actual deformations, by multi-base extensometer data were reasonably close to the predicted deformation analysis through numerical approaches.

Another notable study is by Cai et al. (2004) analysed the stability and deformation behaviour of a large underground cavern in a hard rock mass using 2D FEM. They found that the stability of the cavern was highly dependent on the rock mass properties, support system, and cavern geometry. They recommended using a flexible support system to ensure the stability of the cavern.

Chen et al. (2005) investigated the deformation and stability of a large cavern in the Jinping II Hydropower Station in China using 2D FEM. They found that the stability of the cavern was affected by the geo-mechanical properties of the surrounding rock mass and recommended using a systematic approach to designing the support system for the cavern.

The study by Varadarajan et al. (2007) adds to the body of literature on the behaviour of rock masses during excavation, particularly in the Himalaya region. The study was done at the Nathpa-Jhakri project along Himachal Pradesh using finite element analysis. The study uses strain softening of rock in a model through which the deformation was predicted and finally compared with the observed data obtained from the extensometer which shows that predictions are satisfactory.

Zhang et al. (2008) investigated the deformation and stability of a large cavern in the Xiluodu Hydropower Station in China using 2D FEM. They found that the deformation of the surrounding rock mass was influenced by the geo-mechanical properties, excavation methods and support systems, highlighting the importance of conducting site-specific analyses.

Wong et al. (2009) developed a 2D FEM model to analyse the deformation behaviour of a large underground cavern. The study by Wong et al. (2009) demonstrated the effectiveness of using the finite element method (FEM) to analyse the deformation behaviour of large underground excavations. The study area was the East Rail Extension (ERX) project in Hong Kong, which involved the construction of a 12 km long railway tunnel with multiple underground stations. The authors used the program FLAC to simulate the excavation

process and compared the numerical predictions with extensometer data. They found that the 2D FEM model provided an accurate prediction of the deformation behaviour, and that the comparison with extensometer data was a crucial step in validating the numerical model.

The study by Zhou et al. (2012) focused on investigating the deformation behaviour of large underground openings under different support conditions, using the Jinping II Hydropower Station in China as a case study. The study employed both numerical simulations and field measurements to assess the impact of various support systems on the stability of the underground openings.

In a more recent study, Wang et al. (2014) developed a 2D FEM model to analyse the deformation behaviour of a large underground cavern and compared the numerical predictions with extensometer data. The authors used the program FLAC to simulate the excavation process and validated their model using field measurements. They found that the 2D FEM model provided an accurate prediction of the deformation behaviour, and that the comparison with extensometer data was a crucial step in validating the numerical model.

Winn et al. (2017) analysed the stability analysis of underground storage cavern excavation in Singapore. They carried out and compared the value obtained from FEM analysis with monitored value of vertical displacement of cavern roof and wall during excavation and concluded calculated axial forces were far below design capacity of rock bolts and the excavation were safe throughout the excavation period.

The study by B.K. and Panthee (2019) aimed to investigate the deformation behaviour of underground cavities using numerical methods, with a case study of a tunnel in Rassuwagadhi Hydroelectric Project, Nepal. The study employed a finite element method (FEM) including 2D and 3D FEM to analyse the deformation behaviour of the tunnel, and the results were compared with the extensometer data collected during the construction process.

Another study made by Aghababaei et al. (2019) titled "Numerical modelling of deformation behaviour of an underground cavern and its correlation with multipoint extensometer measurements" investigates the deformation behaviour of a large underground cavern excavated in a hard rock formation. The study employs numerical modelling using the

Phase2 software and correlation with multipoint extensometer measurements for validation and calibration.

The study by Jia et al. (2020) used both 2D FEM and extensometers to analyse the deformation of a powerhouse cavern in Canada. The study found that the deformation of the cavern was influenced by the rock mass properties and the excavation method used. The study also found that the maximum deformation occurred at the crown of the cavern, while the minimum deformation occurred at the invert. The combined use of 2D FEM and extensometer measurements provided a more comprehensive understanding of the deformation behaviour of the cavern.

Shrestha (2021) provide the guideline with illustration for cavern design in phase2 with different stages modelling the ground relaxation applying uniformly distributed load to the excavation profile.

Ghalan (2022) study on the stability assessment of the underground powerhouse for the Tamakoshi V hydroelectric project that focuses on the evaluation of the stability of the powerhouse under various loading conditions, including seismic events, using numerical modelling techniques, and found that the stability of the powerhouse was highly dependent on the strength and deformation characteristics of the surrounding rock mass.

CHAPTER III

MATERIALS AND METHOD

To achieve the goals of the study, a systematic methodology and various tools were utilized, including desk study, field study, laboratory work, numerical modelling, and data processing, interpretation, and report writing. The following methodology was implemented during the research:

3.1 Materials

Several maps, tools, computer programs, and secondary data from various projects were employed for the research study. Numerous materials were used for the study, which is summarized below under various subheadings:

3.1.1 Maps and Data

During the research, the study utilized the topographic map of SOMDAN with sheet number 2885-13 on a 1:50000 scale prepared by the Survey Department, Government of Nepal in 1996 AD. In addition to this, Google images and drone images were used to study drainage patterns, vegetation cover, slope steepness, landscape, and prepare the lithological boundary on the geological map. For the deformation analysis, the study used geotechnical data to determine the rock mass characteristics, such as rock strength, geological conditions, and rock quality designation (RQD). Hydraulic fracturing test data was utilized to obtain stress data, and instrumentation data was collected to measure the strain, displacement, and load applied to the cavern structure.

3.1.2 Equipment

During the fieldwork, several tools were utilized, including the Brunton Compass, geological hammer, hand lens, HCL, measuring tape, GPS, camera, and stationery (Photograph 1). The Brunton compass was employed to measure the attitude of the rock and its discontinuities, while geological hammers were used to assess the strength and weathering condition of the rock. The mineral composition of the rocks was determined by using a hand lens, and the presence of carbonate was identified through diluted HCL (10%). The measuring tape was used to determine the chainage interval and precisely measure rock discontinuities. The GPS

was used to obtain the accurate coordinates of the location and picture were taken using a camera.



Photograph 1 Field equipment used for the study

3.1.3 Computer Software and Program

For data processing, numerical modelling, analysis, interpretation, and report writing, this research uses Google Earth, Microsoft Word, and Excel (2021), Photoshop (2012), AutoCAD 2020, ArcGIS, and Phase2.

3.2 Method

The Methodology adopted in the present study is divided into 5 topics which are listed below:

1. Desk study
2. Field study
3. Laboratory tests
4. Data Compilation, data processing, Numerical Modelling, and Interpretation
5. Report writing

3.2.1 Desk Study

The literature review is taken as the preliminary activity done in this study which includes the following:

- a. Study of the geological status of the study area
- b. Study of the theories currently in practice about the rock mass characteristics and stress-induced instability in weak rock.
- c. Review of available methods for deformation analysis.
- d. Study of the Upper Trishuli-1 Hydroelectric Project.
- e. Overview of the project layout, with a particular focus on the powerhouse complex.

3.2.2 Field Study

The second stage of the research study involved a field trip and the collection of both primary and secondary data. The collection of primary data included:

(a) Geological study

The geological data was collected and analysed to determine the geological status of the study area. Several traverse routes were selected, including Trishuli River, Mailung Khola, different access roads of Upper Trishuli-1, and foot trail to Tiru village. At each location on the traverse route, detailed geological information such as the attitude of foliation plane and lithology was collected with GPS location.

(b) Geotechnical study

The engineering geological data were collected including the rock mass properties of powerhouse cavern. Rock mass classification was done in different chainage intervals of the powerhouse cavern. Brunton compass, a geological hammer, measuring tape, and dilute HCL were used for this purpose. Total five probe hole with length of 8m and 15 m were drilled at the crown of powerhouse cavern and Probe hole log was prepared using the borehole camera. The analysis of cavern roof was made based on the probe hole logging. The different mechanical properties are:

Rock mass Properties

The term "rock mass" refers to rock that has been fractured. Most of the rocks micro to macro fractures form a block together. Discontinuity, strength, deformability, and weathering

condition all affect the quality of a rock mass. The geometry of excavation plays a major role for stability of rocks. The rock mass is characterized by its quality and the mechanical process acting on it (Panthi 2006) and these two plays a crucial role for underground stability as shown in Figure 3 below.

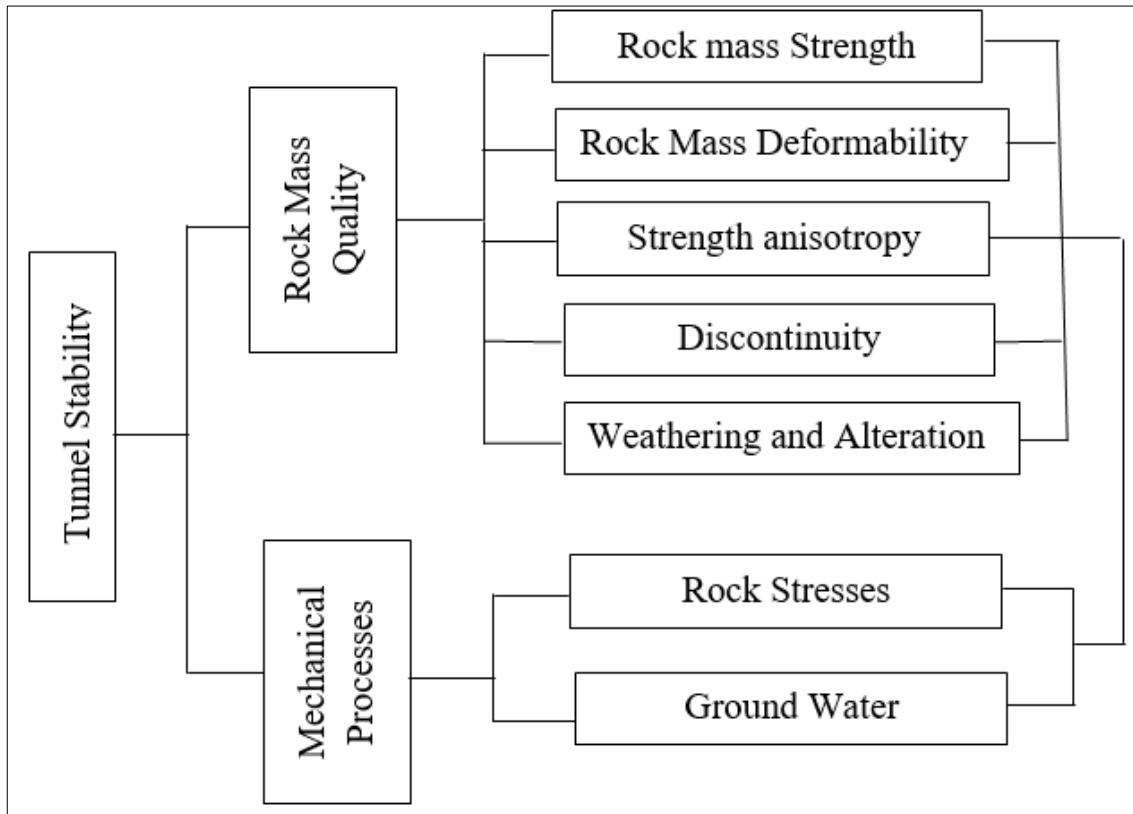


Figure 3 Factors influencing tunnel stability (Panthi 2006)

Rock mass classification is a crucial tool in distinguishing rocks that exhibit similar properties and behaviours. To evaluate the behaviour of the rock mass, it is essential to categorize it into specific groups or classes based on a clearly defined relationship (Bieniaswki 1989). Each group or class is assigned a unique number based on similar properties and characteristics.

Rock Mass Rating (RMR)

Bieniawski (1976) introduced the Geo-mechanics Classification, also known as the Rock mass rating (RMR) system, which is a classification method used for rock masses. This system is used to rates the rock mass quality on a scale of 0 to 100, with higher ratings

indicating good quality of rock mass, and is commonly used in tunnelling. The system has been revised and published in 1989 based on number of case studies.

This rock mass classification system employs six parameters to categorize the rock mass:

- Uniaxial compressive strength of rock material.
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities

The rock mass's strength is evaluated through the Uniaxial compressive strength test, which is conducted in a laboratory. It can also be estimated in the field, but for the present study, the laboratory-tested value is used.

The Rock quality designation (RQD), developed by Deere (Deere et al. 1967) of the rock is the percentage of intact core pieces longer than 100 mm in the total length of the core, in which core should be at least NX size (54.7 mm) in diameter. Palmstrom (1982) suggested that the RQD may be estimated from the number of discontinuities per unit volume. The given relationship for clay free rock masses is:

$$RQD = 115 - 3.3 J_v \quad (1)$$

Where J_v is the volumetric joint count and is defined as the number of joints intersecting a volume of one cubic meter. Where the jointing occurs mainly as joint sets, the following equation was used:

$$J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \dots + \frac{1}{S_n} + \frac{N_r}{5} \quad (2)$$

Where S_1 , S_2 , and S_3 are the average in spacing in meters for the joints sets, and $N_r/5$ is for random joint for spacing of 5 m. This method was used for the determination of RQD in this study.

The spacing of discontinuities is the perpendicular distance between a set of discontinuity. Along with spacing of discontinuities, groundwater conditions were also filled on the RMR

chart. Condition of discontinuities like persistence, aperture, roughness, infill and weathering of the discontinuities and finally orientations of the discontinuities were documented. From the six parameters considered, ratings were allocated and finally the sums of the rating determined the RMR of the rock. The RMR chart is attached in (Appendix A).

Rock Tunnelling Quality Index, Q

The q-system was used as one of the methods for the classification of rock mass during the study. Barton et al. (1974), suggested a Tunnelling Quality Index (Q) system based on different number of studies of underground excavation. The tunnel quality index (Q) value shows the rock mass quality and the stability of a jointed mass in an underground opening. The Q value ranges from 0.001 to a maximum of 1000 and is given as:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad (3)$$

Where, RQD is the Rock Quality Designation, J_r is joint roughness number, J_a is joint alteration number, J_w is joint water reduction factor; and SRF is the stress reduction factor

Here, these given parameter provides the numerical values to be assigned to describe the rock mass condition. The six parameters express the three main factor which describe the stability on underground.

$\frac{RQD}{J_n}$ = Degree of jointing, which basically measures the block size

$\frac{J_a}{J_r}$ = Joint friction or inter-block shear strength, which basically represent the roughness and frictional characteristics of the joint wall and filling material.

$\frac{J_w}{SRF}$ = Active stress,

The tunnel Quality Index Q- chart is attached in (Appendix B)

Geological strength index (GSI)

The geological strength index (GSI) on the study area was calculated using the index given by Sonmez and Ulusay (1999). The GSI uses the ratings such as surface condition rating (SCR) and structure rating (SR). This method of GSI calculation uses volumetric joint count (J_v) and RMR scheme (eg: roughness, weathering, and infilling. Then SR and SCR can be calculated by following equation.

$$SR = -17.5 \ln(J_v) + 79.8 \quad (4)$$

$$SCR = R_r + R_w + R_f \quad (5)$$

Here, J_v is volumetric joint count; R_r , R_w , and R_f are roughness, weathering, and filling parameters of Joint Conditions in RMR.

The geological strength index (GSI) chart is attached in (Appendix C)

Failure Criteria

For the analysis of deformation of powerhouse cavern, the Generalized Hoek and Brown criterion was used. This criterion is best suited for intact rock, or for rock masses with enough closely spaced discontinuities with similar characteristics. Then isotropic behaviour involving failure on discontinuities can be assumed (Hoek, 2007c). The term failure simply means a 'loss of integrity' of the material, which in engineering is interpreted as the loss of load carrying capacity of the materials. Several theories have been developed in order predict and explain when and where failure will occur in the rock mass. This has been done by presuming that a particular mechanism will be responsible for the failure when a particular mechanical property is exceeded (Myrvang 2001). this can also be done by evaluating which principal stress condition will lead to such a failure. Among the classical theoretical failure criteria are the Tresca criterion (maximum shear stress), Mohr-Coulomb (max effective shear stress), Drucker Prager criterion and Griffith's criterion. The theoretical criteria rarely reflect the true nature of the failure mechanism. Many have therefore tried to formulate empirical relationships, where the Hoek-Brown criterion is widely applied. Hoek-Brown criterion are commonly applied in rock engineering that is used as basis in preliminary design calculations and in numerical modelling and will therefore be further explained in the following.

The Hoek-Brown criterion

The Hoek and Brown criterion considers the effect of geological structures, such as joints, bedding planes, and faults, on the strength of the rock mass. This criterion has been widely used for the design of underground excavations and has been applied to a wide range of rock types. The original Hoek-Brown criterion is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left(mb \frac{\sigma_3'}{\sigma_{ci}} + s \right)^{0.5} \quad (6)$$

where σ_1' and σ_3' are the major and minor effective principal stresses at failure,

σ_{ci} is the uniaxial compressive strength of the intact material. m and s are material constants that depends on the rock properties and on the extent to which it has been broken before subjected to the stresses, The rating of the constant m was later adjusted, and the ratio mb/m_i was introduced by Wood (1991) and Hoek et al. (1992). Later Hoek-Brown criterion was extended to generalise Hoek – Brown failure and is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left(mb \frac{\sigma_3'}{\sigma_{ci}} + s \right)^a \quad (7)$$

where the coefficient mb , s and a are the semiempirical parameters that characterize the rock mass, and their relationship is expressed as:

$$m_b = m_i \exp \left(\frac{GSI-100}{28-14D} \right) \quad (8)$$

$$s = \exp \left(\frac{GSI-100}{9-3D} \right) \quad (9)$$

$$a = 0.5 + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right) \quad (10)$$

Where, m and s are the material constant for the intact rock; GSI is the Geological strength index. m_i is the frictional characteristics of the component minerals in rock elements; D is the disturbance factor depending upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed rock masses. The value of constant m_i for intact rock is presented in (Appendix D) and value of Disturbance factor D is given in (Appendix E).

Post-Failure Characteristics

To predict failure mode, it is important to collect information about the elasticity of rock mass. Rock having high elastic modulus are stiff and is naturally brittle. Also, plastic deformation basically occurs with low elastic modulus, forming more deformation. For the study of failure of rock mass, it is essential to estimate post failure behaviour in numerical

modelling. (Hoek and Brown 1997) proposed a post-failure characteristic for a rock of varying quality while carrying out modelling for studying rock mass behaviour after failure, as shown in Figure 4 below.

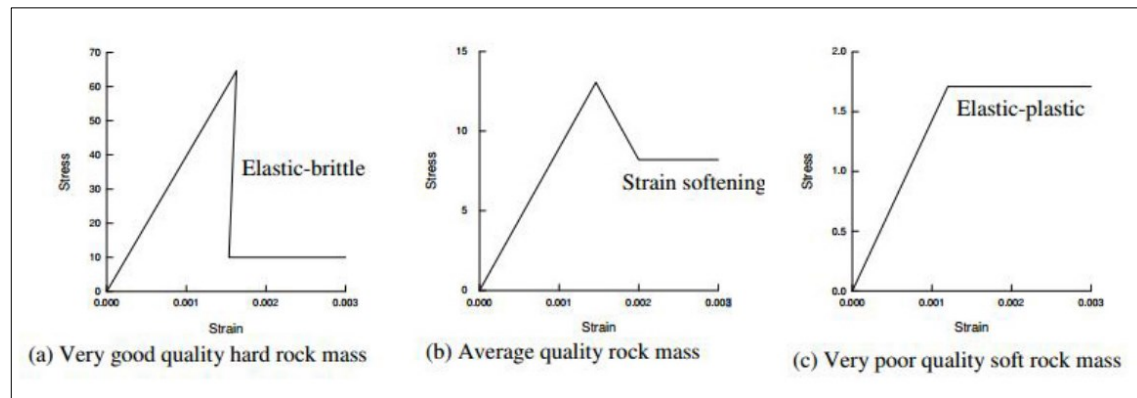


Figure 4 Post Failure Characteristics of rock based on quality (Hoek and Brown, 1997)

Estimation of rock mass deformability

Young's modulus, often known as the modulus of elasticity (E), is the ratio between applied stress and corresponding strain within the elasticity limit and describes the deformability of intact rock. When discussing the deformability of rock mass, the term "modulus of deformation" (E_m) refers to the ratio of stress to corresponding strain during loading, which includes both elastic and inelastic behaviour. Since a jointed rock mass does not act elastically, the term "modulus of deformation" is used. Modulus of deformation can be tested directly in the field using techniques such as plate bearing, dilatometer test, flat jack test, and hydraulic chamber etc. These tests are time consuming and expensive so, many authors have therefore proposed empirical equations for estimating the modulus of deformation. During the study, the equations based on GSI, RMR and Q were used for calculating deformation modulus as given in Table 1 below.

Table 1 Empirical formulas for the estimation of rock mass deformation modulus

Purposed by	Empirical relationship
Bieniawski (1978) E_m (Gpa) for $RMR > 50$	$E_m = 2RMR - 100$
Serafim and Pereira, 1983 E_m (Gpa) for $RMR < 50$	$E_m = 10^{\left(\frac{RMR-10}{40}\right)}$
Palmstrom and Singh (2001)	$E_m = 8 * Q^{0.4}$

Hoek and Diedrichs (2006)	$\frac{E_m}{E_r} = 0.02 + \left(\frac{1-D/2}{1 + e^{(60+15D-GSI)/11}} \right) \text{ (Gpa)}$
---------------------------	---

Where,

E_m is in-situ deformation modulus of rock mass in GPa,

RMR is rock mass rating system,

Q is Tunnel Quality Index,

GSI is Geological strength index,

σ_{ci} is the uniaxial compressive strength of the intact rock materials, D is a variable that depends on the level of blast damage and stress relaxation that the rock mass has experienced. It ranges from 0 for in situ rock mass that is not disturbed to 1 for highly disturbed rock mass. The value of D is presented on (Appendix E).

3.2.3 Laboratory Work

Core samples were collected from the field for laboratory work. The Uniaxial compressive strength (UCS) of rock samples of powerhouse cavern was done in the laboratory of Upper Trishuli-1.

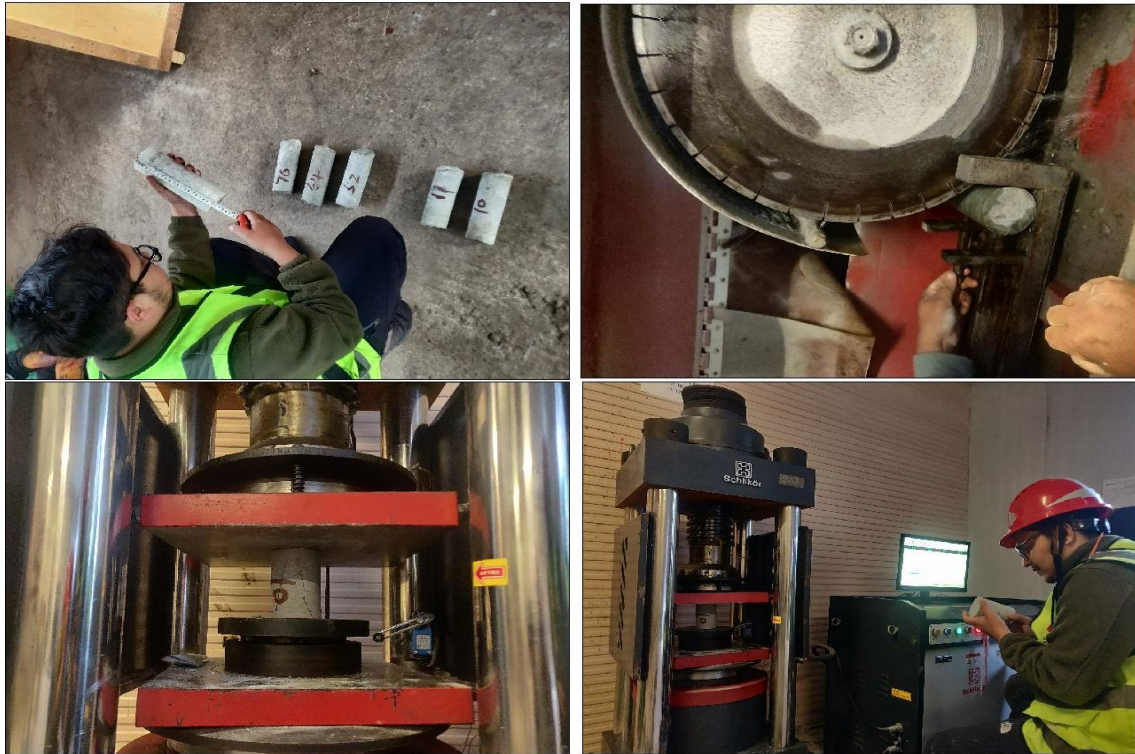
Uniaxial Compressive Strength Test

The uniaxial compressive test is a crucial laboratory technique that determine the strength of rock and other geological materials subjected to compression. This method involves applying a uniaxial load or force to a cylindrical sample of the material until it fractures. The compressive strength of any geological material is the measured force which is necessary to cause failure. The core sample were collected from the Upper Trishuli-1 which was recovered from the Powerhouse Cavern and the sample was tested in sophisticated Uniaxial Compressive strength testing machine (photograph 2).

IS 9143 suggested the method for determining uniaxial compressive strength of rock material. The length to diameter ratio of cylindrical specimen shall preferably be 2 to 3. The unconfined compressive strength of the specimen shall be calculated by dividing the maximum load carried by the specimen during the test, by the average original cross-sectional area.

$$UCS = \frac{P}{A} \quad (11)$$

Where, P is maximum axial load applied to the rock sample at failure, and A is the cross-sectional area of the rock sample.



Photograph 2 Uniaxial compressive strength test in laboratory

3.2.4 Data Compilation, Data Processing, Numerical Modelling, and Interpretation

The data collected during the field work and laboratory work were compiled, refined, to prepare a model and were simultaneously analysed and interpreted using various computer tools and software such as Microsoft Excel, ArcGIS, Google earth, Photoshop, Coral Draw AutoCAD and Phase2.

After the completion of field work, geological map was prepared using ArcGIS 10.5 and map were used for interpretation of geological conditions. The engineering geological map of powerhouse cavern were used for analysis of geological and geotechnical condition. Further the data from of multipoint extensometer (secondary data provided by Upper Trishuli-1) was assembled in excel sheet and graph of Deformation (in Y axis) Verses Date

(in X- axis) were plotted and interpretation was done. The location of extensometer installed position of the powerhouse cavern is shown in (Appendix F)

Numerical modelling

To analyse the stress and deformation of powerhouse cavern of Upper Trishuli-1 FEM based numerical analysis was adopted. FEM is a widely used numerical method for analysing the behaviour of underground caverns. This method divides the rock mass into a mesh of small elements and uses mathematical equations to simulate the behaviour of each element under various loading conditions. It is a powerful method that can accurately model complex geometries and non-linear material behaviour. The phase2 software is a comprehensive and powerful software which is applied to conduct the calculation using the finite element method (FEM). One of the key features of Phase2 software is its ability to model tunnelling and excavation design capabilities. Phase2 consists of three basic program modules:

- i) Model
- ii) Compute
- iii) Interpret

Model

Model is the first task to define in Phase2. Various input parameters should be given in the model. which includes the loading condition, material properties, support properties and discontinuity properties. During the study, the real ground elevation (1250m) was used for the external boundary. Total of 27 stages was created and mesh type is selected as graded with element type three noded triangles as shown in Figure 5 below. The loading condition was taken as constant because of having a cavern not nearer to the surface. The material model is taken as Hoek and Brown for the analysis as it is widely used for modelling and analyzing the intact or jointed rock mass. This model considers the effect of rock strength, jointing and in-situ stress, jointing on its deformation behaviour. The excavation sequence of powerhouse cavern was provided by Upper Trishuli-1 which is shown in (Appendix G). The support system is used at powerhouse cavern with the safety factor of 1.2 for shotcrete and 1.5 for rock bolt. The data related with support system are provided by Upper Trishui-1 is given in (Appendix H, I, J).

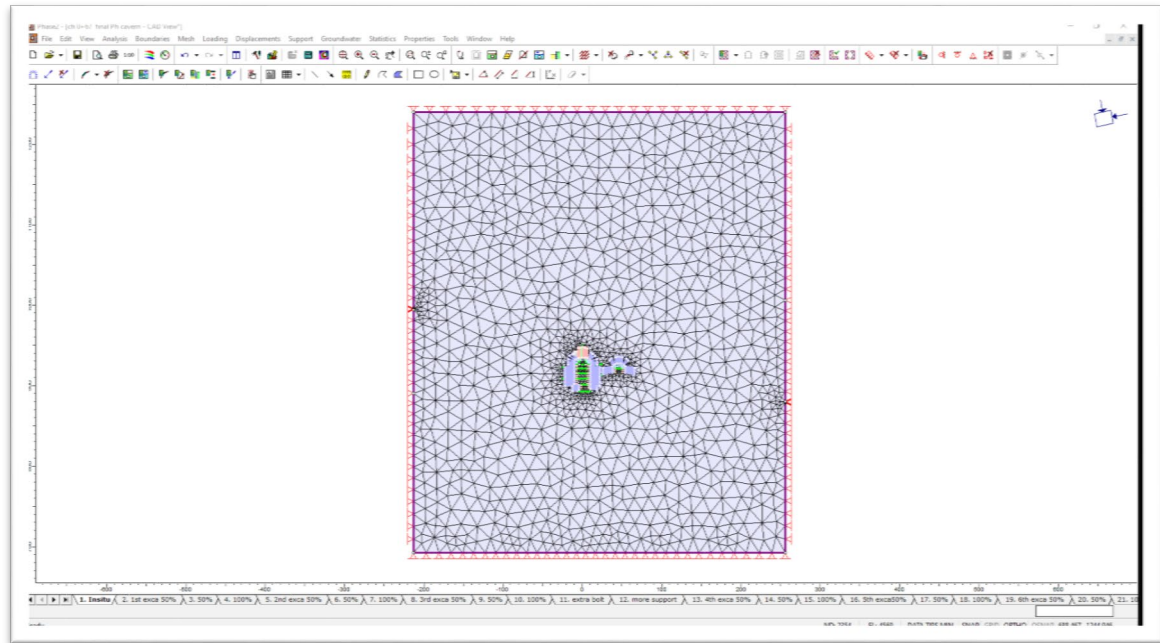


Figure 5 Mesh generated with 3 noded triangles

In Phase2, the stage sequence comprising the excavation, installation of rock supports, and stress release are as follows.

For chainage 0+67

- Stage 1- In-situ stress condition (no excavation)
- Stage 2- Central heading excavation of powerhouse cavern + 50% stress release
- Stage 3- 50% stress release + 200 mm 25 MPa shotcrete lining+ 8 m long 32 mm dia. rock bolt.
- Stage 4- 100% stress release.
- Stage 5- Left heading excavation of powerhouse cavern + 50% stress release
- Stage 6- 50% stress release + 200 mm 25 MPa shotcrete lining + 8 m long 32 mm dia. rock bolt +8 m long 32 mm dia. rock dowel.
- Stage 7- 100% stress release.
- Stage 8- Right heading of powerhouse cavern + 50% stress release
- Stage 9- 50% stress release + 200 mm 25 MPa shotcrete lining + 8 m long 32 mm dia. rock bolt +8 m long 32 mm dia. rock dowel.
- Stage 10- 100% stress release.
- Stage 11- 10 m rock dowel (inclined) on the crown +12m vertical rock dowel on central crown

- Stage 12- Bench down of the powerhouse cavern from EL. 926.50 to EL. 921.50, as well as transformer cavern from EL. 929.10 to EL. 921.50. + 50% stress release on both caverns
- Stage 13- 50% stress release + (160 mm shotcrete lining +8 m long 32 mm dia. rock bolt) on transformer cavern crown and (160 mm shotcrete lining + 12 m long 32 mm dia. rock dowel) on powerhouse cavern walls.
- Stage 14- 100% stress release on both caverns.
- Stage 15- Bench down of the powerhouse cavern from EL. 921.50 to EL. 915.70 as well as bench down the transformer cavern + 50% stress release on both caverns
- Stage 16- 50% stress release + (160 mm shotcrete lining + 8 m long 32 mm dia. rock dowel) on transformer cavern and (160 mm shotcrete lining + 12 m long 32 mm dia. rock dowel) on powerhouse cavern walls.
- Stage 17- 100% stress release on both caverns.
- Stage 18- Bench down of the powerhouse cavern from EL. 915.70 to EL. 908.50 as well + 50% stress release
- Stage 19- 50% stress release + (160 mm shotcrete lining + 12 m long 32 mm dia. rock dowel) on powerhouse cavern walls.
- Stage 20- 100% stress release on Powerhouse cavern.
- Stage 21- Bench down of the powerhouse cavern from EL. 908.50 to EL. 901.30 + 50% stress release on powerhouse cavern
- Stage 22- 50% stress release 160 mm 30 MPa shotcrete lining +12 m long 32 mm dia. rock dowel on walls of Powerhouse cavern.
- Stage 23- 100% stress release on Powerhouse cavern.
- Stage 24- Bench down of the powerhouse cavern from EL. 901.30 to EL. 896.10 + 50% stress release on powerhouse cavern
- Stage 25- 50% stress release + 160 mm shotcrete lining + 8 m long 32 mm dia. Rock dowel on walls of powerhouse cavern.
- Stage 26- 100% stress release.

For the input parameters for rock deformation analysis in Phase2 (1) loading conditions, (2) material properties, (3) support properties and (4) joint properties are given. In loading condition, the field stress type constant stress is selected because of the powerhouse cavern being deeper from surface where there is no effect of topography in the magnitude and

direction of the stress. The field stress is taken as initial element loading in the material properties and the material type is taken as plastic and generalized Hoek and Brown criterion is used as a failure criterion for the analysis of deformation. At the end, the support was installed on the model. The support data was provided by the Upper Trishuli-1.

Compute

Once the modelling was finished, the next step was computation of the model where the maximum iteration is selected as 500 with 0.001 tolerance as shown in Figure 6.

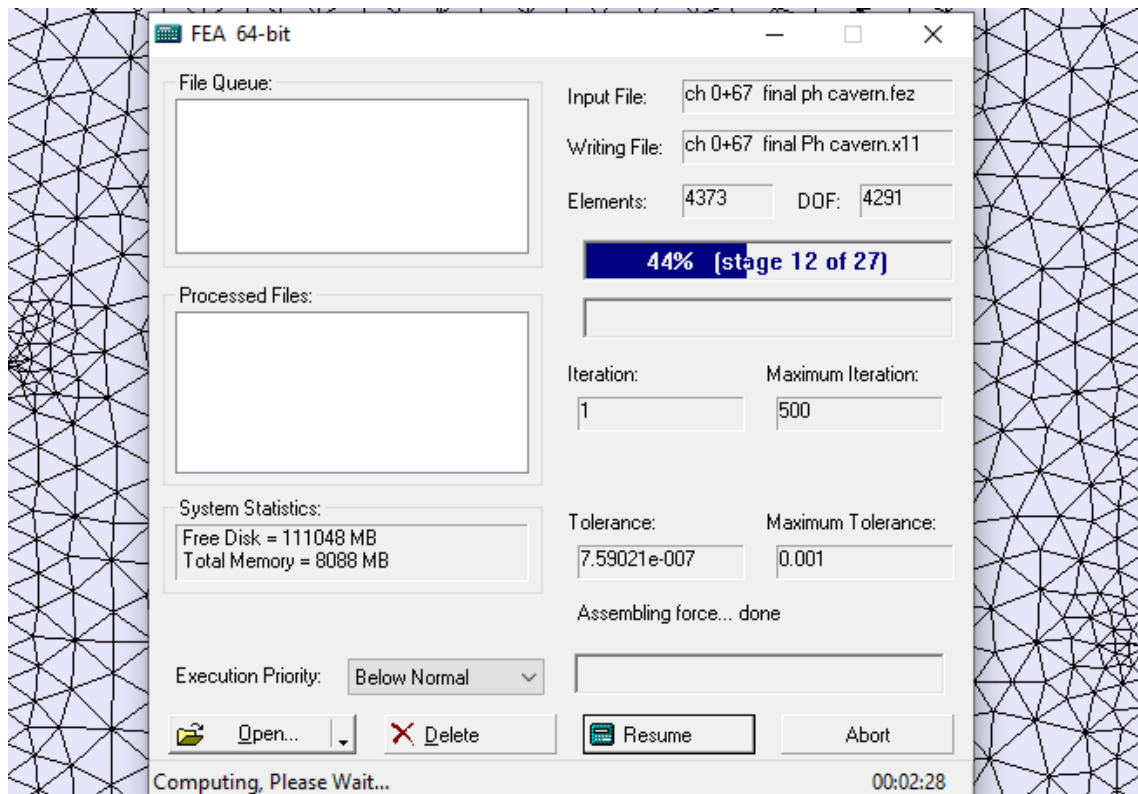


Figure 6 Computation of model using Finite element analysis

Interpretation of results

Finally, analysis was made and, it is possible to visualize and display stress trajectories, yield in rock mass and support, support capacity of reinforced concrete, location, and progress of vertical, horizontal, and total displacements etc. Displacement profiles along the excavation boundary can be used to compare the model results to measured cavern deformation.

CHAPTER-IV

RESULTS

Detailed geological mapping and traverse were conducted to achieve the research objectives. The geological mapping around the Upper Trishuli-1 Powerhouse area of the Mailung, Tiru, and Gogane area of the Rasuwa district is described in the following section.

4.1 Geology of the study area

The study area is mapped under the Kuncha Formation of Nawakot Complex from Central Nepal and the Ulleri Gneiss as the intrusion of the Kuncha Formation rocks (Figure 7). The geological units of the study area are categorized into the Kuncha Formation and Ulleri Gneiss as follows.

a. Kuncha Formation

The name of this geological unit is adapted after Stocklin and Bhattarai (1977). In the study area, the rock consists of interbedding of the light grey, medium-grained, moderately foliated psammitic schist, and greenish grey, fine-grained Pelitic schist observed at the Powerhouse area. The rocks are composed of fresh to slightly weathered, medium-thick beds of psammitic schist and slightly to moderately weathered, thinly foliated pelitic schists (Photograph 3).

This type of rock is also well-exposed around the confluence of the Trishuli River and the Mailung Khola. This geological unit also consists of the soapy touch wavy phyllite, and grey quartzite with quartz veins at and around the Tiru village (Photograph 4). In the study area, the Kuncha Formation has occupied the southern and northern portion.

b. Ulleri Augen Gneiss

During the geological traverse along the Mailung Khola, Ulleri Augen Gneiss are well exposed at about 800 m upstream from the confluence of the Mailung Khola and Trishuli Rivers (Photograph 5). This type of rock is also observed on the road traverse from Mailung Bajar to Tiru village and Tiru to Gogane village and towards Trishuli Rivers. The rocks are bounded by the rocks of the Kuncha Formation. The rocks are composed of grey to white, thick to the massive bed of the augen gneiss. The rocks show clear bands of quartz, feldspar,

and mica in variable quantities. This geological unit is hosted within the Kuncha Formation, the oldest unit of the Nawakot Group in the Lesser Himalaya. The geological map is shown below at Figure 7.

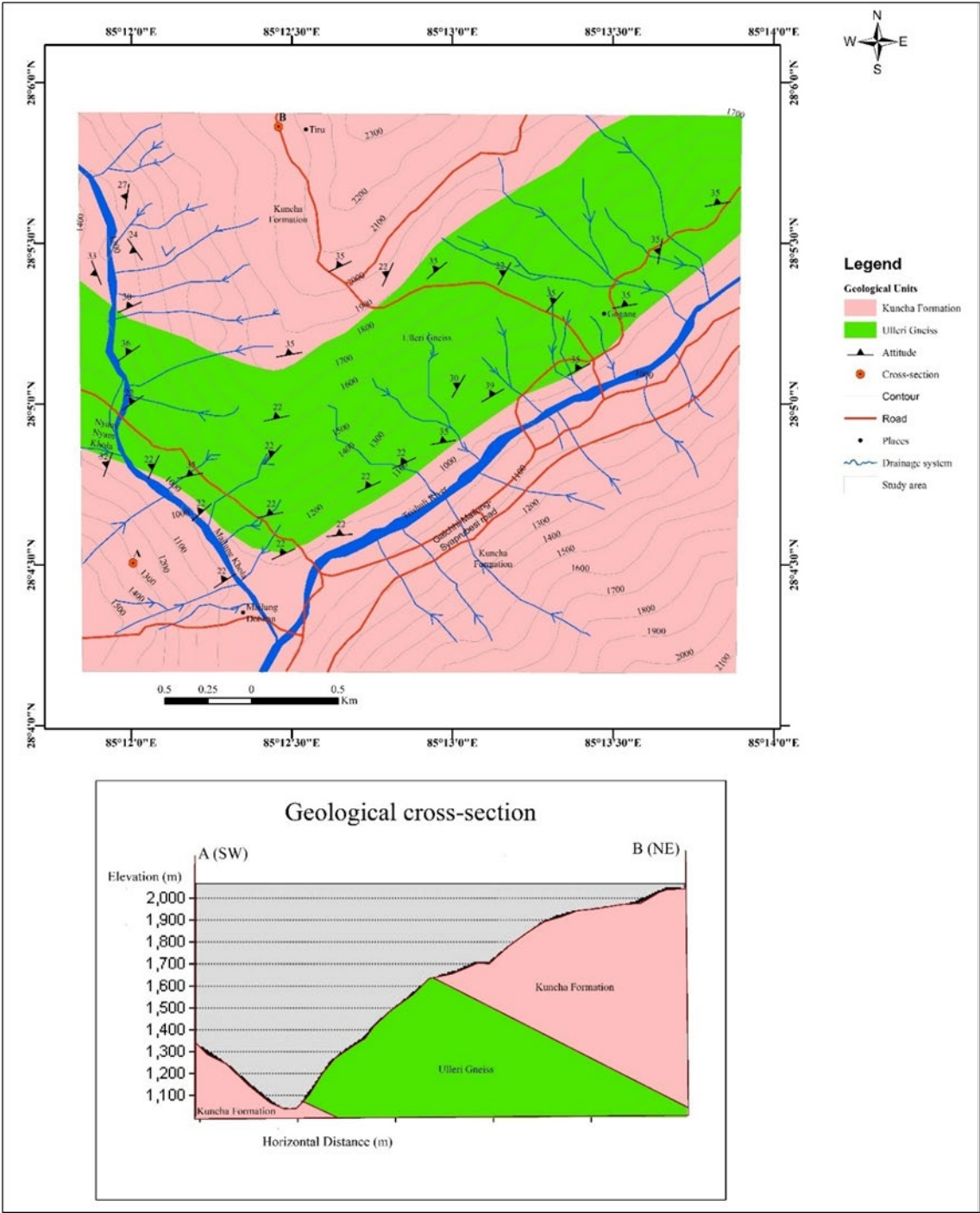


Figure 7 Geological map of study area



Photograph 3 Interbedding of the psammitic schist and pelitic schist observed in the Upper Trishuli-1 Powerhouse Cavern at the confluence of the Trishuli River and Mailung Khola



Photograph 4 Interbedding of the wavy phyllite with quartzite rocks observed 500m south of the Tiru valley



Photograph 5 Thick to a massive bed of the Ulleri Augen Gneiss observed at about 1 km from Mailung Bajar along the traverse of Mailung to Tiru village



Photograph 6 The geological contact of the Kuncha Formation and the Ulleri Augen Gneiss observed at about 500 m north of the Powerhouse of the project Upper Trishuli-1

4.1.1 Engineering geological condition of powerhouse cavern

The powerhouse cavern consists of light grey medium to fine-grained, medium strong to strong, moderately foliated, psammitic schist with interbedding and parting of greenish grey, weak to medium strong, thinly foliated, pelitic schist (Photograph 7). The degree of weathering of the powerhouse cavern is slightly weathered state to moderately weathered state. Most of the discontinuities were in a tight to a wide aperture, showing medium persistence with very close to wide spacing. The roughness was represented as ranging from a slightly rough state to an undulating state, and the infilling on the joint opening consists of little clay to no filling. The overall geometry of rock mass is tabular and irregular. From the geological mapping (Figure 8) and stereographic analysis of the Cavern, the four groups of joints (Photograph 8) with the most frequent distribution and their properties are given in Table 2 below.



Photograph 7 Distribution of schist unit at powerhouse cavern

The geological mapping of powerhouse cavern was done after every excavation sequence and the data are compiled together forming a as-built map as shown in Figure 8. A number of discontinuity data are measured at each 20m interval of different chainage.

Table 2 Description of major discontinuities of powerhouse cavern

J1	Foliation	350-10/12-30	spacing is 0.1-0.3m, the local spacing is 0.01-0.05m, the persistence > 2m, the surface is rough and undulating
J2	joint	125-145/59-74	spacing is >20cm, the persistence is 1-5m and some >5m, the surface is rough and undulating
J3	joint	355-15/60-85	spacing is >10cm to a few meters, the persistence 1-24m, the surface is rough and undulating
J4	joint	170-195/62-78	spacing is 0.2-1m, the persistence 0.5-8m, the surface is rough and undulating



Photograph 8 Combination of potentially unstable structural surfaces on the downstream side wall of the powerhouse cavern

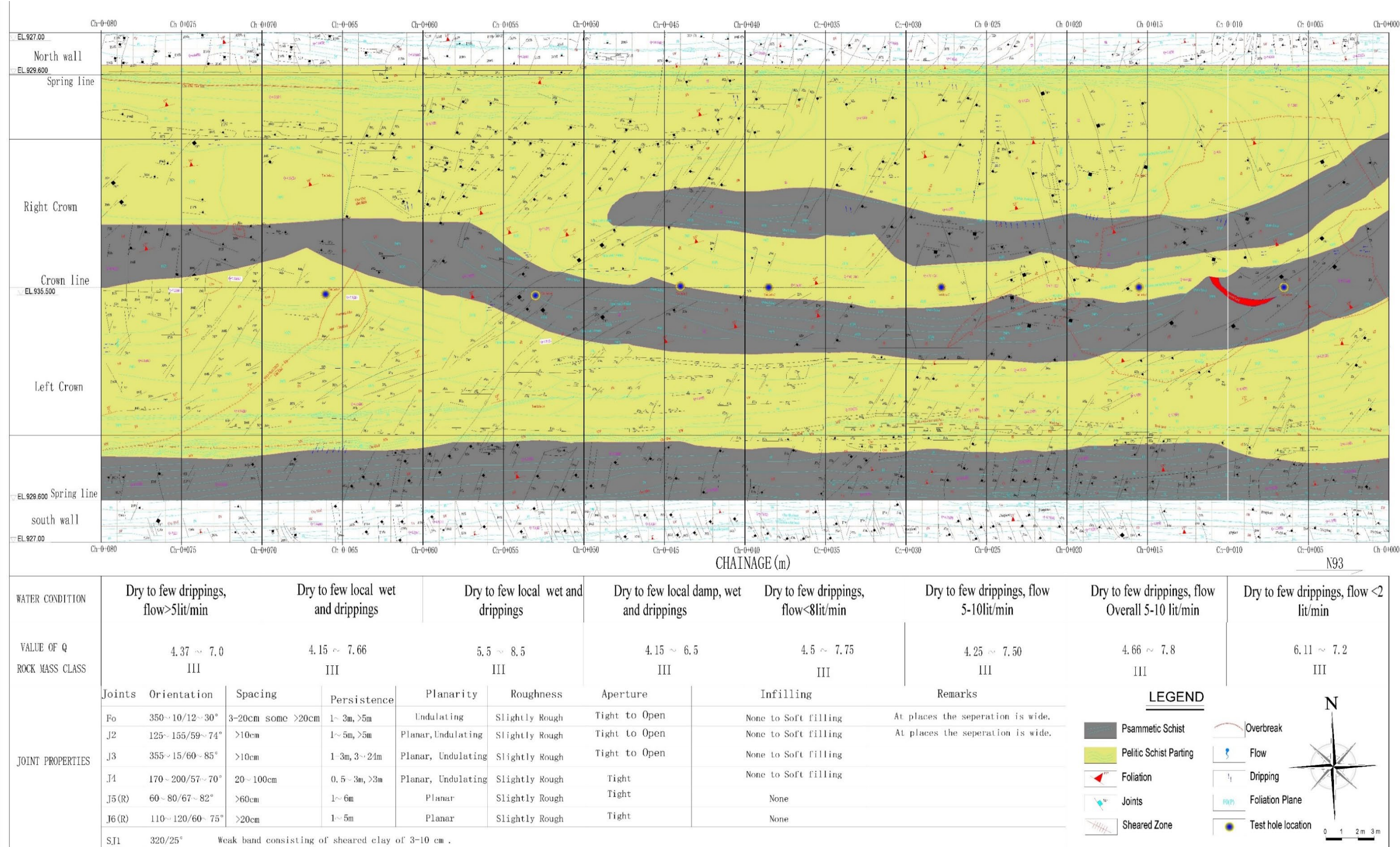


Figure 8 As-built geological map of Powerhouse caver

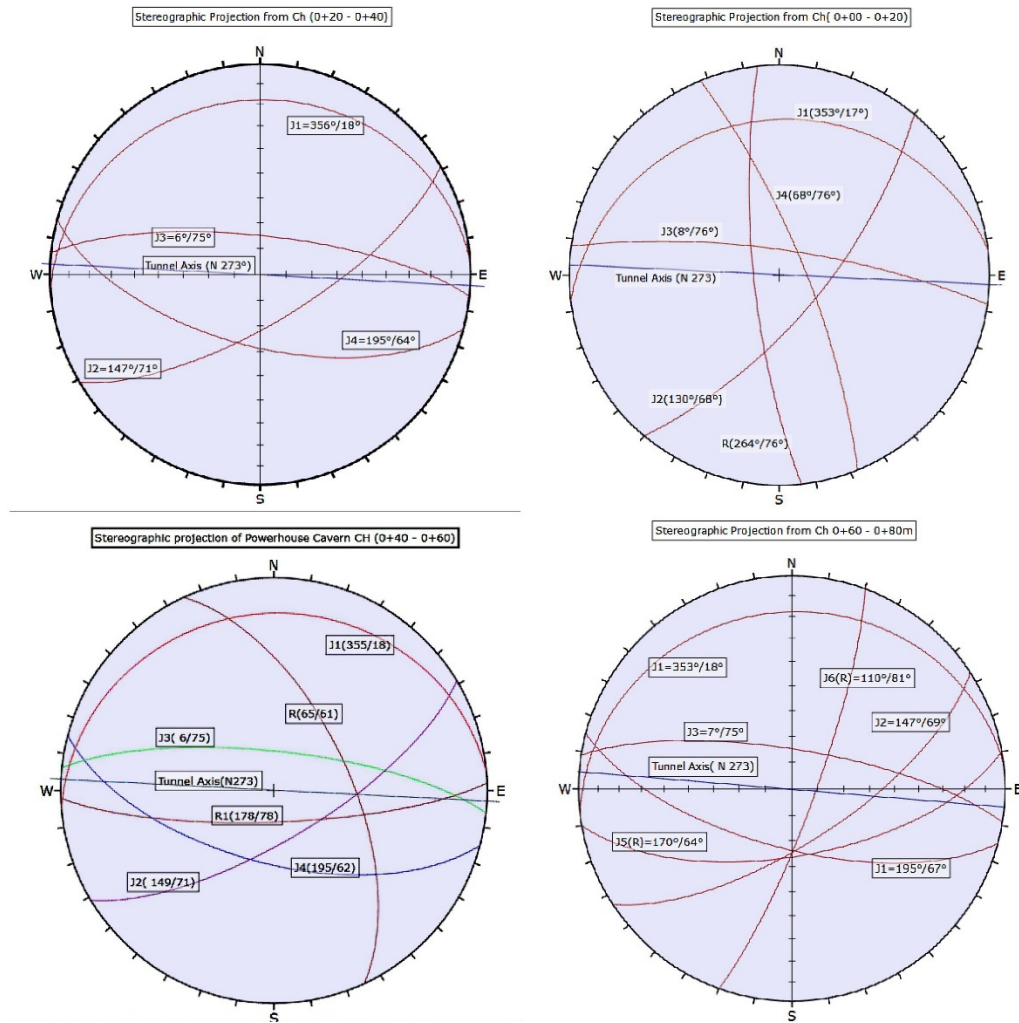


Figure 9 Stereographic Projection of discontinuities measured form Powerhouse cavern

Here, Figure 9 shows the stereographic projection of discontinuities of powerhouse cavern. These Hemispherical plots are prepared every 20m section along the Powerhouse cavern. The combination of these joints forms tetrahedral and prismatic wedge along the powerhouse crown and wall (Photograph 8). Altogether, six probe holes were drilled at the central crown in order to understand the geological condition of the Powerhouse cavern roof. The log (Figure 10) was prepared at a scale of 1:50 and from the study, the 2m section of the cavern roof consists of a disturbed rock mass. There is an uneven distribution of quartz veins which is mostly associated with pelitic Schist. A small clay band associated with the weak rock was observed at the depth of 6.7m. The powerhouse crown consists of interbedding of psammitic schist and pelitic schist. Few open joints are observed at the crown. Figure 10 below shows the probe hole log of the powerhouse crown. From the probe hole data, the area between 5m to 10m depth have numerous weak bands and quartz vein associated with the weak band.

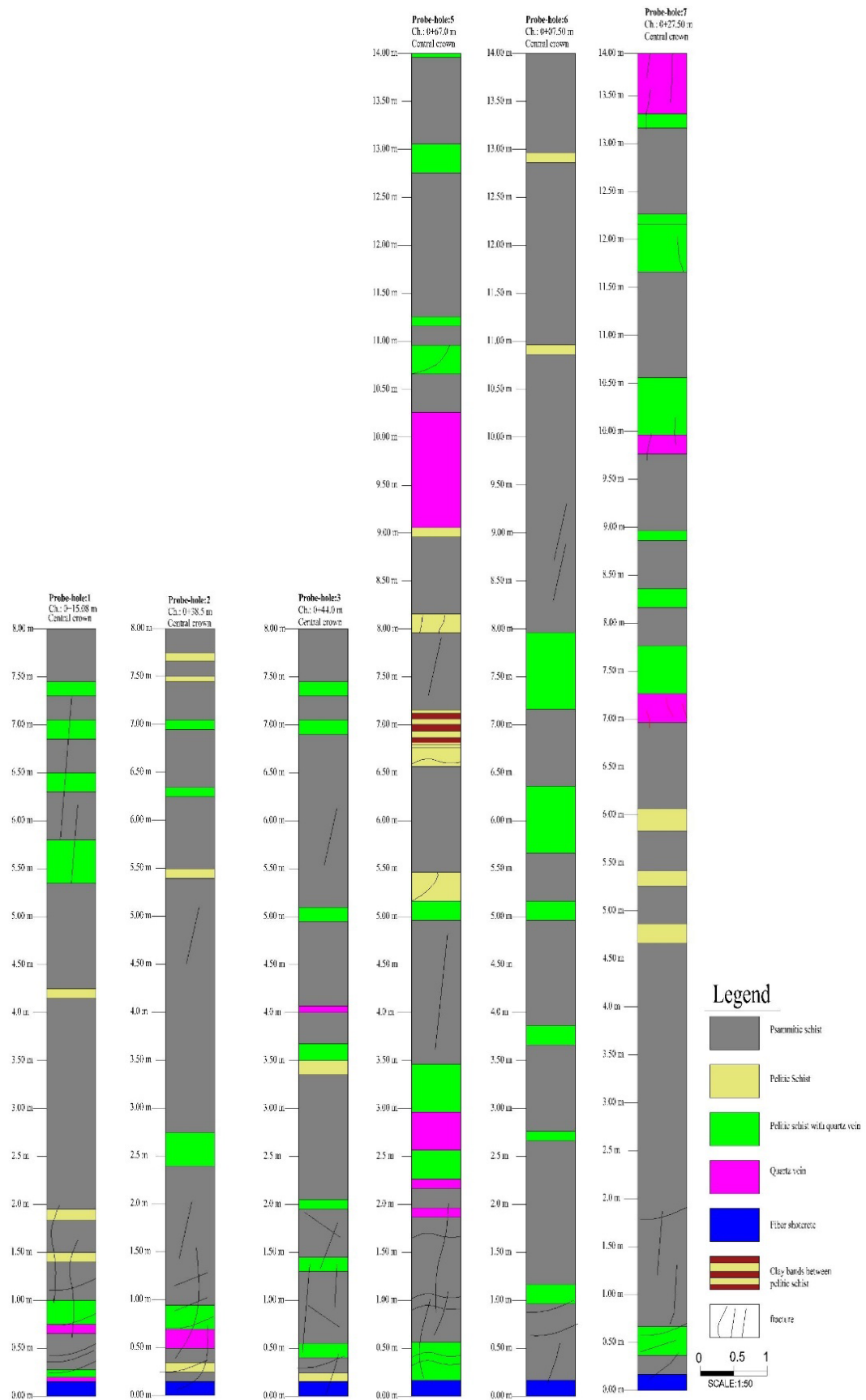


Figure 10 Probe-hole log of Powerhouse Cavern crown

4.2 Geotechnical properties of rock mass

Geotechnical studies of the underground structures include the establishment of geotechnical parameters to know the existing ground condition and the structure built on the ground i.e., field and laboratory techniques which describe the characteristics of the rock mass condition and geological condition of the area under study. This chapter includes geo-mechanical classification of rock mass RQD, RMR, Q, GSI, and geotechnical studies of in-situ deformation modulus, rock mass behaviour, and rock mass strength parameters. The required parameters were taken in the field and estimated from various empirical relations.

4.2.1 Estimation of rock mass quality

Various rock mass classification systems were adopted for the present study are, Rock quality designation RQD (Palmstrom 1982), Bieniawski's geomechanics classification system RMR (1989), Tunnel Quality Index Q (Barton et al. 1974), and Geological strength index GSI. (Sonmez and Ulusay 1999). The rock mass classification has been carried out along the alignment of the powerhouse cavern at a 5 m chainage interval from the west wall to the east wall. These geo-mechanical classification values were used to calculate in-situ deformation modulus and rock mass strength parameters and these parameters were also used in numerical modelling.

Rock mass rating

Rock mass ratings were done in the field after every excavation cycle and made a sum up after the completion of the first layer. At first, the central heading was done and the rock mass classification was done accordingly. The estimated rock mass was used to calculate the in-situ deformation modulus of the rock mass. The RMR value ranges from 46 to 57 which comes under rock class III shown in the Table 3 below:

RQD, GSI, and Q

The rock quality designation, GSI, and Q were done at different chainage intervals of 5m using the method developed by Palmstrom (1982), Bieniawski (1989), Grimstand and Barton (1933) and Hoek and Brown (1997). The calculated value of RQD is 52 to 72 and the calculated value of GSI ranges from 39 to 52 (Table 4). The Q-value ranges from 4.15 to 7.21 (Table 5).

The overall rock mass classification shows the rock mass of the powerhouse cavern is classified as rock class III (fair rock) on the basis of all classification systems as shown in Table 6 below. Similarly, the distribution of RMR, GSI, Q and RMR from Q system is given in Figure 11 below. The distribution of these different classification system shows small changes rock mass where RMR from Q estimates maximum and GSI and RMR gives closer values.

Table 3 Summary of RMR along the underground powerhouse cavern (UT1)

Chainage	Strength of intact rock material	RQD	Spacing of Discontinuities	Persistence	Aperture	Roughness	Infilling	Weathering	Groundwater Condition	Orientation	RMR
0+00 - 0+05	7	13	8	2	3	3	4	5	10	-2	53
0+05 - 0+10	7	13	10	2	3	3	4	5	7	-2	52
0+10 - 0+15	7	13	10	3	3	3	4	5	4	-2	50
0+15 - 0+20	7	13	8	2	3	3	4	5	4	-2	47
0+20 - 0+25	7	13	10	3	3	3	4	5	10	-2	56
0+25 - 0+30	7	13	8	4	3	3	4	5	10	-2	55
0+30 - 0+35	7	13	10	2	3	3	4	5	10	-2	55
0+35 - 0+40	7	13	8	2	3	3	4	5	10	-2	53
0+40 - 0+45	7	13	8	2	3	3	4	5	10	-2	53
0+45 - 0+50	7	13	8	2	1	3	2	5	10	-2	49
0+50 - 0+55	7	13	8	2	1	3	2	5	10	-2	49
0+55 - 0+60	7	13	8	2	3	3	4	5	10	-2	53
0+60 - 0+65	7	13	8	2	3	3	4	5	10	-2	53
0+65 - 0+70	7	13	8	2	1	3	2	5	7	-2	46
0+70 - 0+75	7	13	8	2	3	3	2	5	10	-2	51
0+75 - 0+80	7	13	10	2	3	3	6	5	10	-2	57

Table 4 Summary of GSI along the underground powerhouse cavern (UT1)

Chainage	Volumetric Joint Frequency	Structural Rating (SR)	Roughness Rating (Rr)	Weathering Rating (Rw)	Infilling Rating (Rf)	Surface Condition Rating	GSI
0+00 - 0+05	16	31	3	5	4	12	45
0+05 - 0+10	13	35	3	5	4	12	47
0+10 - 0+15	13	35	3	5	4	12	47

Chainage	Volumetric Joint Frequency	Structural Rating (SR)	Roughness Rating (Rr)	Weathering Rating (Rw)	Infilling Rating (Rf)	Surface Condition Rating	GSI
0+15 - 0+20	14	34	3	5	4	12	45
0+20 - 0+25	15	32	3	5	4	12	50
0+25 - 0+30	15	32	3	5	4	12	50
0+30 - 0+35	12	36	3	5	4	12	45
0+35 - 0+40	13	35	3	5	4	12	47
0+40 - 0+45	14	34	3	5	4	12	45
0+45 - 0+50	16	31	3	5	2	10	40
0+50 - 0+55	19	28	3	5	2	10	39
0+55 - 0+60	16	31	3	5	4	12	44
0+60 - 0+65	16	31	3	5	4	12	44
0+65 - 0+70	16	31	3	5	2	10	40
0+70 - 0+75	15	32	3	5	2	10	41
0+75 - 0+80	14	34	3	5	6	14	52
Average	14.81	32.74	3.0	5.0	3.63	11.6	45.6

Table 5 Summary of Q-Index along the underground powerhouse cavern (UT1)

Chainage	Volumetric Joint Frequency (Jv)	RQD	Joint Set Number (Jn)	Joint Roughness Number (Jr)	Joint Alteration Number (Ja)	Joint water Reduction (Jw)	Stress Reduction Factor (SRF)	Q-value
0+00 -	16	62.2	6	3	2	1	2.5	6.22
0+05 -	13	72.1	6	3	3	1	2.5	4.81
0+10 -	13	72.1	9	3	2	1	2.5	4.81
0+15 -	14	68.8	6	3	3	1	2.5	4.59
0+20 -	15	65.5	9	3	2	1	2.5	4.37
0+25 -	15	65.5	9	3	2	1	2.5	4.37
0+30 -	12	75.4	6	3	3	1	2.5	5.03
0+35 -	13	72.1	6	3	2	1	2.5	7.21
0+40 -	14	68.8	9	3	2	1	2.5	4.59
0+45 -	16	62.2	9	3	2	1	2.5	4.15
0+50 -	19	52.3	6	3	2	1	2.5	5.23
0+55 -	16	62.2	9	3	2	1	2.5	4.15
0+60 -	16	62.2	6	3	3	1	2.5	4.15
0+65 -	16	62.2	6	3	3	1	2.5	4.15
0+70 -	15	65.5	9	3	2	1	2.5	4.37
0+75 -	14	68.8	6	3	3	1	2.5	4.59

Table 6 Distribution of RMR, GS and RMR from Q system

Chainage(m)	Jv	RQD	GSI	Q-Value	RMR	RMR from Q
0+00 - 0+05	16	62.2	45	6.22	53	60
0+05 - 0+10	13	72.1	47	4.81	52	58
0+10 - 0+15	13	72.1	47	4.81	50	58
0+15 - 0+20	14	68.8	45	4.59	47	58
0+20 - 0+25	15	65.5	50	4.37	56	57
0+25 - 0+30	15	65.5	50	4.37	55	57
0+30 - 0+35	15	65.5	45	5.03	55	59
0+35 - 0+40	13	72.1	47	7.21	53	62
0+40 - 0+45	14	68.8	45	4.59	53	58
0+45 - 0+50	16	62.2	40	4.15	49	57
0+50 - 0+55	19	52.3	39	5.23	49	59
0+55 - 0+60	16	62.2	44	4.15	53	57
0+60 - 0+65	16	62.2	44	4.15	53	57
0+65 - 0+70	16	62.2	40	4.15	46	57
0+70 - 0+75	15	65.5	41	4.37	51	57
0+75 - 0+80	14	68.8	52	4.59	57	58

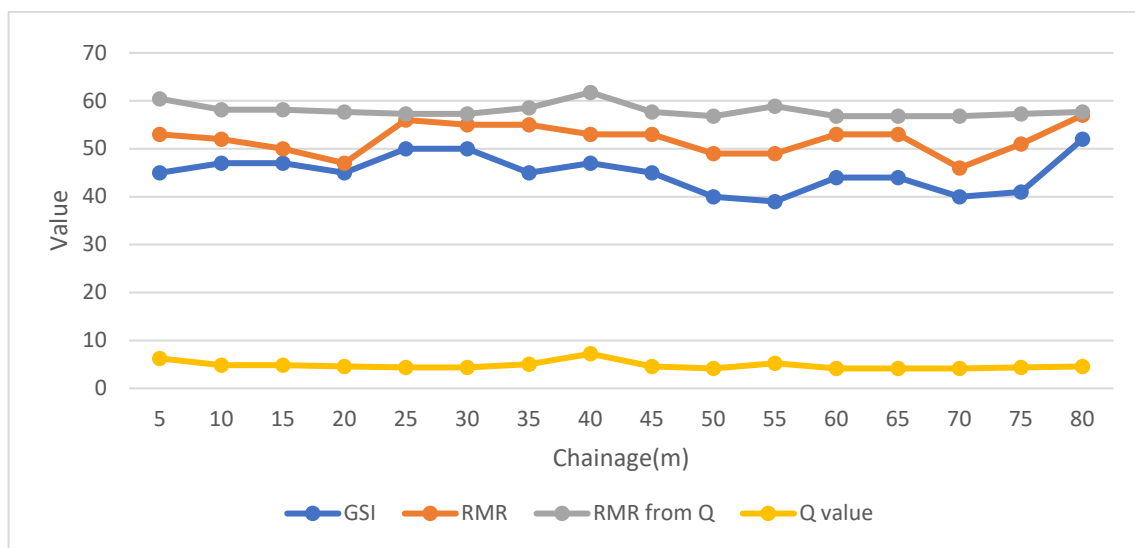


Figure 11 Distribution of RMR, GSI, and RMR from Q along the powerhouse cavern

4.2.2 Estimation of rock mass strength

Rock mass strength refers to the ability of a rock mass to resist external forces and maintain its stability. It is influenced by several factors, including the inherent strength of the individual rock units, the geometry and orientation of the rock units, the presence of fractures, faults, and joints, and the stress conditions in the rock mass. The strength of rock mass can be quantified through various methods, such as laboratory testing, field testing and

numerical modelling.

a. Laboratory test

The Laboratory testing involves testing small samples of rock to determine their strength properties. The geotechnical properties of the rock mass of the powerhouse area were determined in the laboratory. Core samples were taken from the powerhouse cavern of Upper Trishuli-1.

Uniaxial Compressive Strength test

Six core samples were taken for the uniaxial compressive strength test of rock material from the powerhouse cavern. IS-1943 suggested a method was adopted for determining the uniaxial compressive strength test. The UCS value ranges from 38.96 to 81.03 MPa with an average UCS of 60.85 MPa. Table 7 shows the result obtained from the test.

Table 7 Calculation of uniaxial compressive strength of rock material

S.N.	Sample No.	Rock Type	Failure load (KN)	Cross Section Area(mm²)	Uniaxial Compressive strength (MPa)
1	S10	psammitic schist with the parting of pelitic schist	246	3850	63.89
2	S11		235	3850	61.03
3	S34		185	3850	48.05
4	S52		150	3850	38.96
5	S64		278	3850	72.20
6	S76		312	3850	81.03
Average UCS=60.85 MPa					

b. Empirical method

The rock mass strength parameters were estimated using the Hoek –Brown failure criteria. These parameters are essential for providing the rock mass strength which later is used for the numerical as an input parameter as shown in Table 8 below.

Table 8 The rock strength parameters along the powerhouse cavern are presented below

Chainage	GSI	UCS (σ_c) MPa	D	mi	mb	s	a
0+00 - 0+05	45	60	0.3	10	0.992	0.0011	0.509
0+05 - 0+10	47	60	0.3	10	1.079	0.0014	0.507
0+10 - 0+15	47	60	0.3	10	1.079	0.0014	0.507
0+15 - 0+20	45	60	0.3	10	0.992	0.0011	0.509
0+20 - 0+25	50	60	0.3	10	1.224	0.0021	0.506

Chainage	GSI	UCS (σ_c) MPa	D	mi	mb	s	a
0+25 - 0+30	50	60	0.3	10	1.224	0.0021	0.506
0+30 - 0+35	45	60	0.3	10	0.992	0.0011	0.509
0+35 - 0+40	47	60	0.3	10	1.079	0.0014	0.507
0+40 - 0+45	45	60	0.3	10	0.992	0.0011	0.509
0+45 - 0+50	40	60	0.3	10	0.804	0.0006	0.512
0+50 - 0+55	39	60	0.3	10	0.771	0.0005	0.513
0+55 - 0+60	44	60	0.3	10	0.951	0.0010	0.509
0+60 - 0+65	44	60	0.3	10	0.951	0.0010	0.509
0+65 - 0+70	40	60	0.3	10	0.804	0.0006	0.512
0+70 - 0+75	41	60	0.3	10	0.838	0.0007	0.511
0+75 - 0+80	52	60	0.3	10	1.331	0.0027	0.505

c. Estimation of in-situ deformation modulus

The deformation modulus was calculated using the empirical relations given by Bieniawski (1978), Serafim and Pereira (1983), Palmstrom and Singh (2001), and Hoek and Diederichs (2006) using Disturbance factor as 0.3 and 0.5 for the stability analysis of the powerhouse cavern. The calculated value is presented in Table 9.

Table 9 Estimation of in-situ deformation modulus (E_m) from RMR, Q, and GSI

Chainage	Q-value	Palmstrom and Singh (2001)	GSI	For D=0.5	For D=0.3	RMR	Bieniawski, 1989 E_m (Gpa),	Serafim and Pereira, 1983 E_m (Gpa) for
				Hoek and Diederichs (2006) GPa	Hoek and Diederichs (2006) GPa			
0-00 to 0-05	6.2	16.6	45	2.12	2.86	53	6	11.9
0-05 to 0-10	4.8	15	47	2.41	3.27	55	10	13.3
0-10 to 0-15	4.8	15	47	2.41	3.27	57	14	15
0-15 to 0-20	4.6	14.7	45	2.12	2.86	48	-4	8.9
0-20 to 0-25	4.4	14.4	50	2.93	3.98	55	10	13.3
0-25 to 0-30	4.4	14.4	50	2.93	3.98	57	14	15
0-30 to 0-35	5	15.3	45	2.12	2.86	53	6	11.9
0-35 to 0-40	7.2	17.6	47	2.41	3.27	57	14	15
0-40 to 0-45	4.6	14.7	45	2.12	2.86	53	6	11.9
0-45 to 0-50	4.1	14.1	40	1.53	2.05	48	-4	8.9
0-50 to 0-55	5.2	15.5	39	1.44	1.92	48	-4	8.9
0-55 to 0-60	4.1	14.1	44	1.98	2.68	48	-4	8.9
0-60 to 0-65	4.1	14.1	44	1.98	2.68	46	-8	7.9
0+65 to 0-70	4.1	14.1	40	1.53	2.05	48	-4	8.9
0-70 to 0-75	4.4	14.4	41	1.63	2.19	52	4	11.2
0-75 to 0-80	4.6	14.7	52	3.34	4.53	55	10	13.3
Average (E_m)	4.8	14.9	45.06	2.1875	2.956875	52.1	4.1	11.5

d. Field stresses

The In-situ stress of the powerhouse is based on the report on hydraulic fracturing tests. The whole process of measurement is carried out in strict accordance with the American ASTM-D-4645 standard and the methods recommended by international rock mechanics. The test result used in this research work is (secondary data) provided by Upper Trishuli-1 Hydroelectric Project.

Hydraulic fracturing tests have been conducted in the Ventilation tunnel. Three boreholes are drilled from the floor down to a depth of 30 m, left and right wall and the depth of burial is 305 m and the conclusions are based on the 3D stress results. The maximum principal stress is horizontal and the intermediate principal stress is vertical which is greater than the minimum principal that is generally $SH > SV > Sh$. The test result is summarized in Table 10.

Table 10 Hydraulic fracturing tests result, Project data

Location	Principal stress	Value (MPa)	Azimuth angle	Dip angle	Components of Stress (MPa)	
Ventilation tunnel	σ_1	7.81	19.57	11.27	$\sigma_x=7.26$	$\zeta_{xy}=-1.27$
	σ_2	5.99	239.01	75.53	$\sigma_y=4.36$	$\zeta_{yz}=7.26$
	σ_3	3.83	111.37	8.96	$\sigma_z=6.01$	$\zeta_{xz}=4.36$

(Source: UpperTrishuli1 Hydroelectric Project)

4.2.3 Adopted parameters for numerical analysis

The adopted mechanical parameters for the numerical analysis are summarized in Table 11. The powerhouse cavern and a part of transformer cavern are used to compute the model. The input parameters are selected using the empirical method, laboratory test and field observation. Some input parameters used in the model including the field stress data was provided by Upper Trishuli-1.

Table 11 Mechanical parameters adopted for numerical analysis of powerhouse caverns

Chainage	GSI	Unit weight (γ)(kN/m ³)	Elastic modulus of Intact Rock (Gpa)	Elastic modulus of Rock mass (Gpa)	Poisson's ratio of rock mass(μ)	Strength Parameters			
						UCS of intact rock MPa	Peak		
							mb	s	a
0+39	47	26.5	20	3.27	0.25	60	1.07	0.00143	0.507
0+67	40	26.5	20	2.05	0.25	60	0.8	0.00060	0.511

4.3 Numerical method

Deformation and stress analysis by numerical methods

Two-dimensional models were used for the analysis of stress and displacement of the powerhouse cavern. The layout of the powerhouse longitudinal axis is EW direction. The distribution of stress and the deformation behaviour is predicted from Phase2 8.0. Few geotechnical parameters adopted in the model are provided by the Upper Trishuli-1. The modelling conditions are a plane strain. The material law is elastoplastic combined with strain-softening (for rock). The rock mass is modelled as a continuum, homogeneous and isotropic. The failure criteria adopted in the model are generalized Hoek and Brown criteria as it is best applicable for the intact rock and rock mass with the discontinuities closely spaced and have a similar characteristics and isotropic behaviour (Hoek 2007). The 2m section from the excavation line was taken as a disturbed zone after analysing the condition of the Powerhouse cavern from probe hole camera logging (Figure 9) and, assuming that the effects of excavation of rock mass is sensitive and will be confined within the region. In contrast, for the tunnel, the disturbed zone is set at 1-2m with a disturbance factor of $D=0.5$ (Hoek and Brown, 2019). The initial element loading is taken as field stress only with constant field stress type as the cavern lies far and deep from the slope. The maximum overburden of the cavern is 315m from the surface of the excavation. The deformation analysis was performed in two different chainages one is at Ch. (0+39) m and another is at Ch. (0+67) m. The total displacement of the cavern opening on its boundary and outside the boundary is 2 m, 5 m, 10, and 17 m which is equal to the depth of the anchor of the multipoint extensometer.

4.3.1 Stress distribution after excavation and support at Ch 0+39 m

The stress distribution of the powerhouse cavern was performed using the Phase2 software which is based on the finite element method. The major principal stress σ_1 distribution at Ch 0+39 crown along the extensometer points is given in Figure 12. The value of σ_1 is constant at in-situ conditions and gradually changes with the excavation sequences as shown in Figure 13. At the first layer of excavation, σ_1 varies between 3.2 MPa to 10.99 MPa and gradually increases to the final layer of excavation. The σ_1 distribution is maximum at the crown and lower part of the invert and minimum at the side wall and ranges from 0.65MPa to 14.45MPa after the final layer excavation.

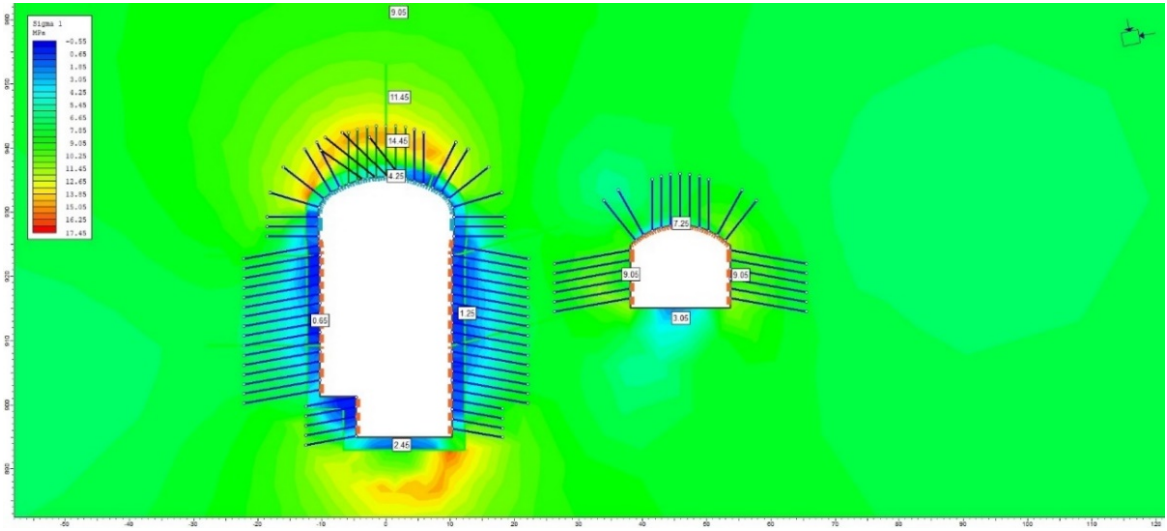


Figure 12 Distribution of sigma 1 stress magnitude at chainage 0+39m

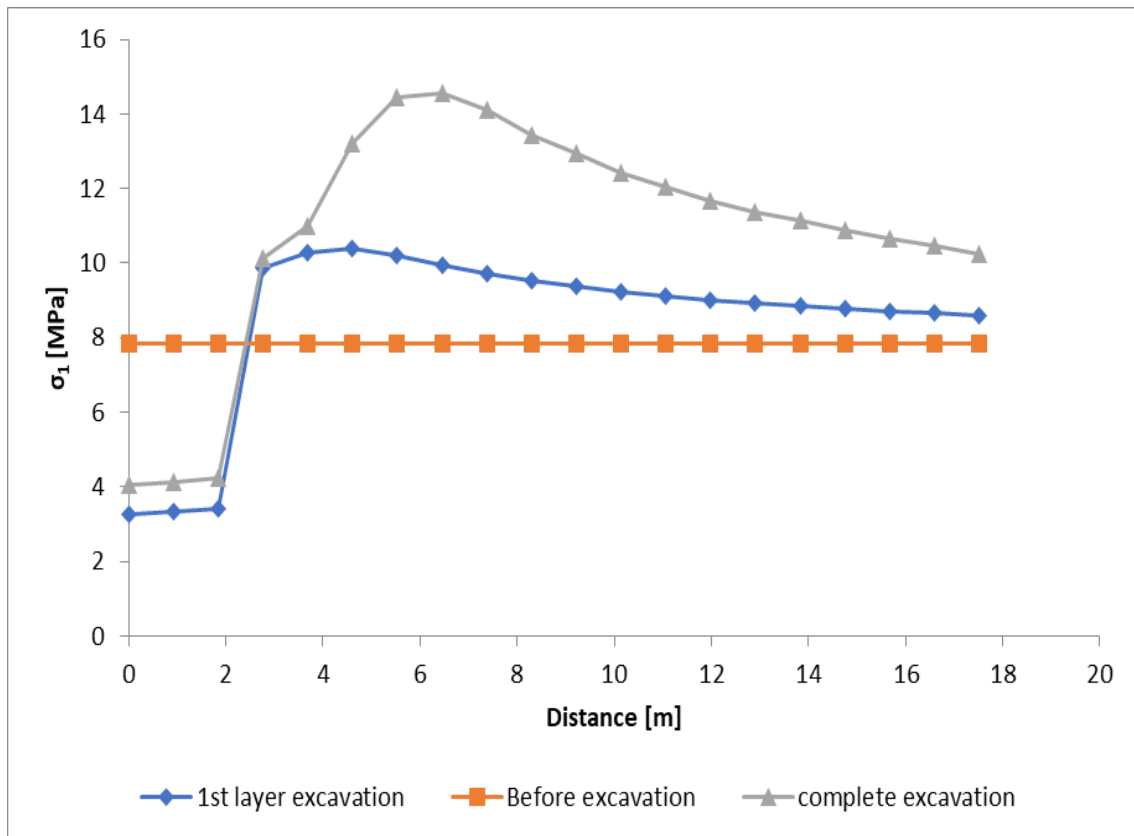


Figure 13 Distribution of sigma 1 stress magnitude at chainage 0+39m

Also, the distribution of σ_3 ranges from 0.50MPa to 6.25MPa as shown in Figure 14. The distribution of the magnitude of σ_3 decreased along the excavation line after the complete excavation. The distribution of the magnitude of σ_3 stress at chainage 0+39m before excavation, during, and after excavation is shown in Figure 15 below.

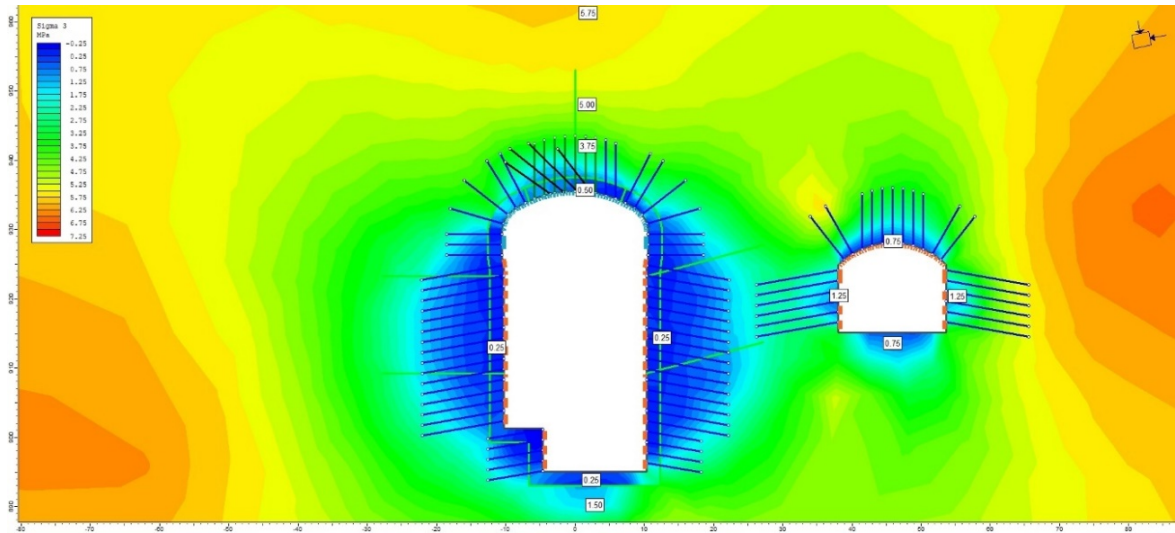


Figure 14 Distribution of sigma 3 stress magnitude at chainage 0+39m

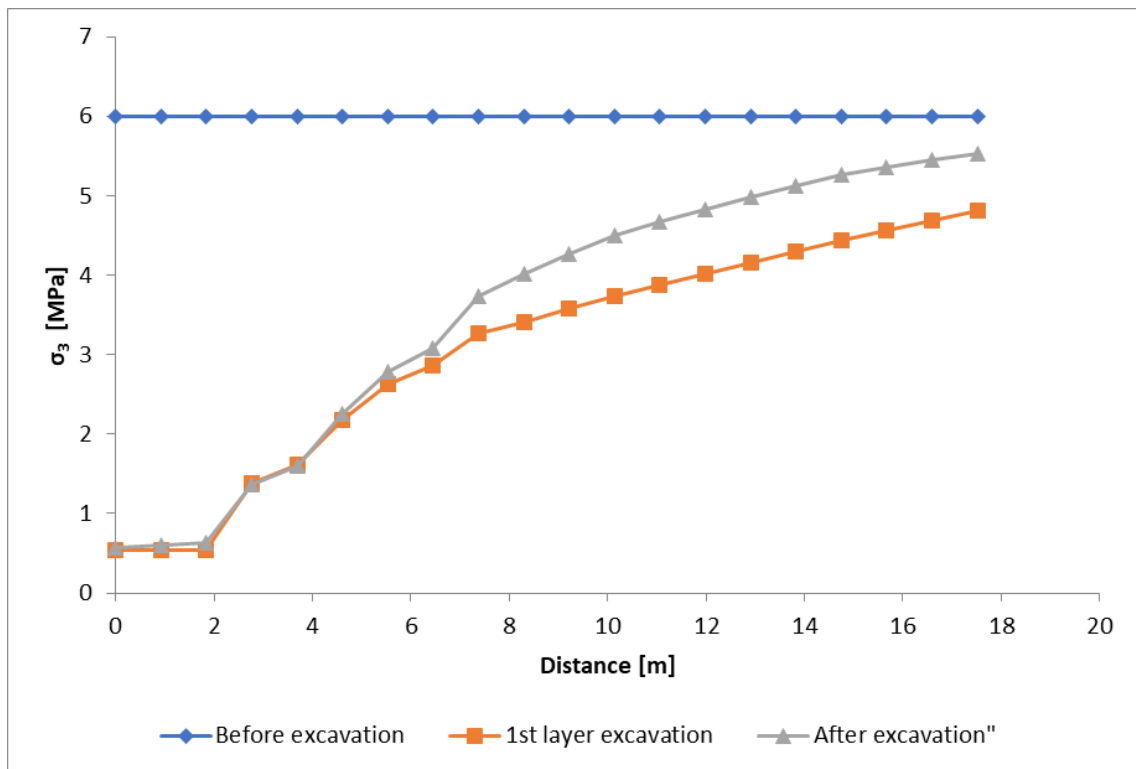


Figure 15 Distribution of sigma 3 stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation.

Similarly, the distribution of mean stress ranges from 0.75 MPa to 7.75 MPa as shown in Figure 16. The distribution of the magnitude of mean stress decreased along the excavation line after the complete excavation. The mean stress increased at the top and bottom of the

excavated surface. The distribution of the magnitude of mean stress at chainage 0+39 m before excavation, during, and after excavation is shown in Figure 17 below.

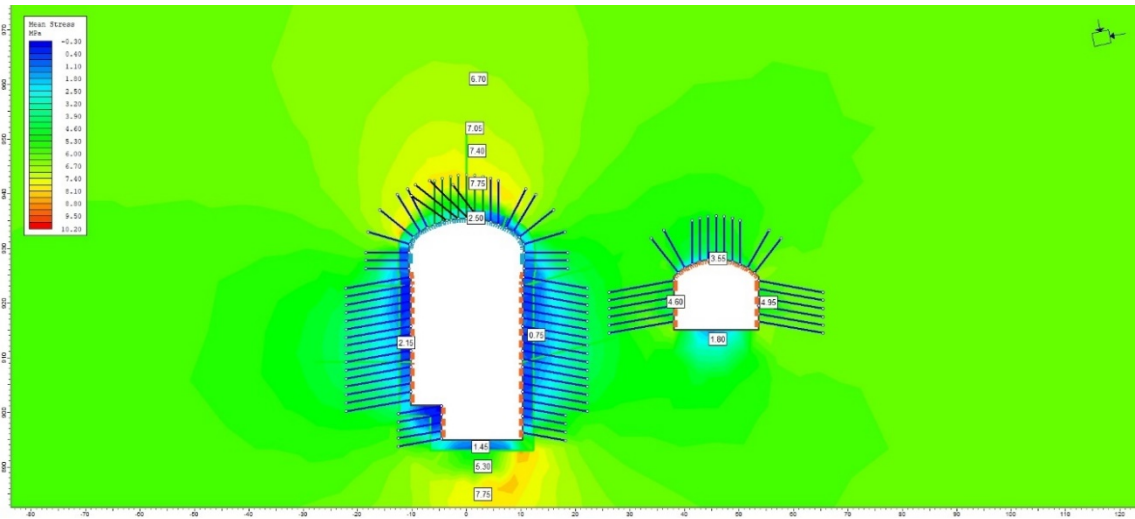


Figure 16 Distribution of mean stress magnitude at chainage 0+39m

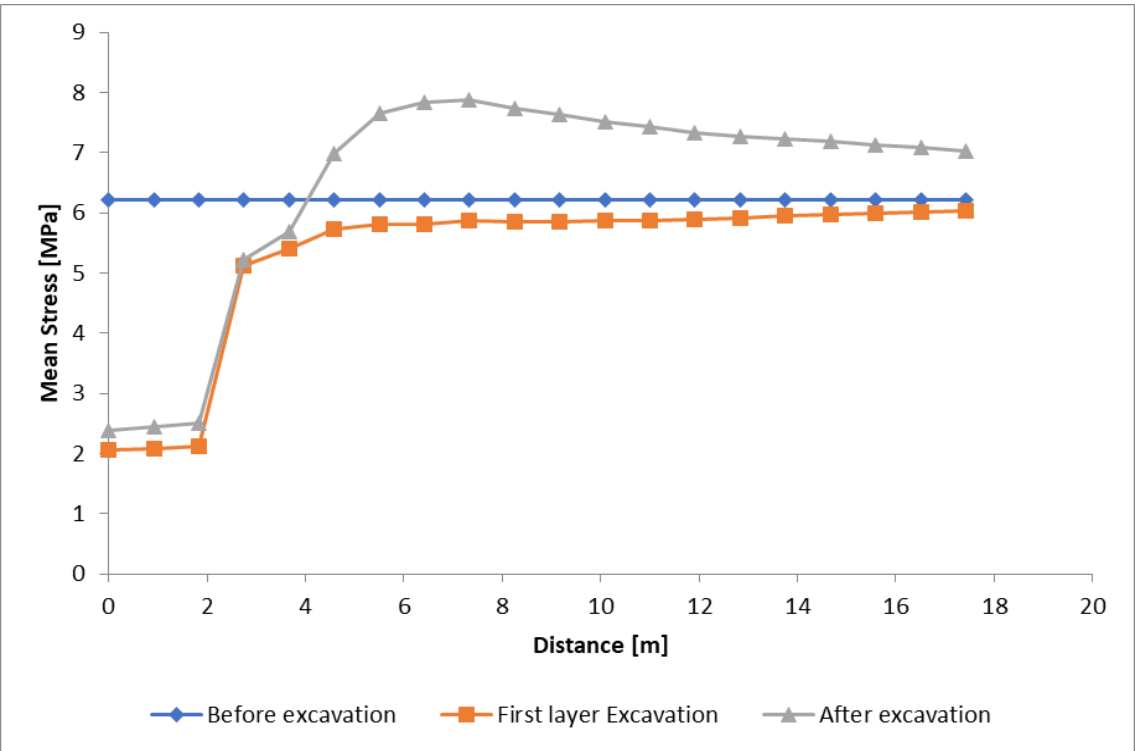


Figure 17 Distribution of mean stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation

4.3.2 Deformation of Powerhouse Cavern at Ch 0+39 m

According to the FEM calculation result, the displacement of monitoring points (Appendix F) at each excavation stage is shown in Figure 18. The total relative deformation on the crown at Ch 0+39 after the first excavation and support along the crown is maximum than at side walls. The displacement at the crown is 52 mm, 36 mm, 24 mm, 19 mm, and 11 mm at 0m, 2m, 5m, 10m, and 17m respectively.

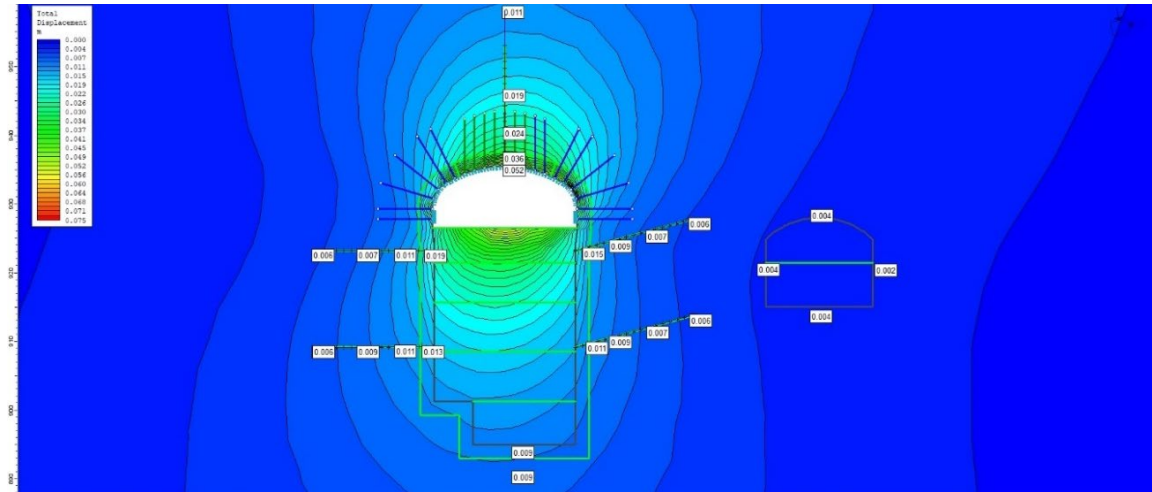


Figure 18 Displacement of monitoring points after first layer excavation

From the model, the total deformation on the crown at Ch 0+39 after the second layer excavation and support is 55 mm, 37 mm, 26 mm, 21 mm, and 13 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The upstream wall has a maximum deformation after the second layer excavation which is 55 mm as shown in Figure 19 below.

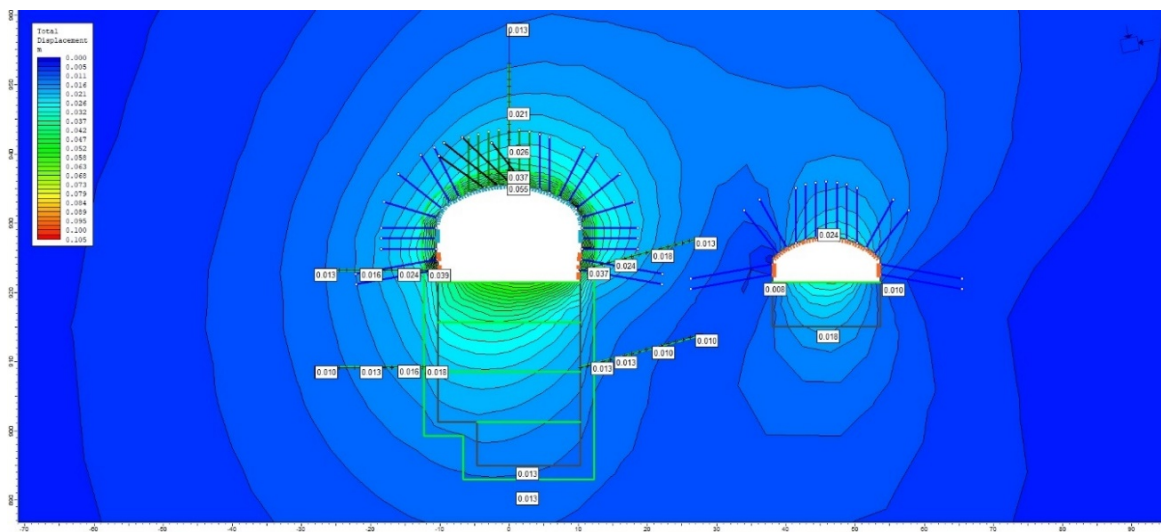


Figure 19 Displacement of monitoring points after second layer excavation

Similarly, the total deformation on the crown at Ch 0+39 from the model after the third layer excavation and support is 60 mm, 42 mm, 27 mm, 21 mm, and 15 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after third layer excavation is at the upstream wall and downstream wall which is 66 mm as shown in Figure 20 below. The second layer of the transformer cavern was excavated simultaneously.

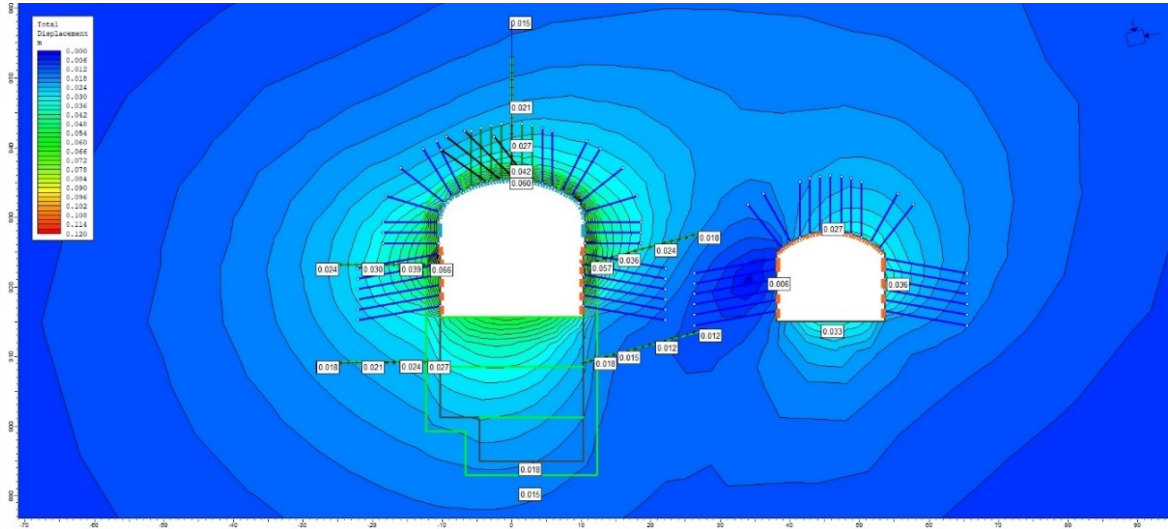


Figure 20 Displacement of monitoring points after third layer excavation

Also, the total deformation on the crown at Ch 0+39 after the fourth layer excavation and support is 60 mm, 40 mm, 28 mm, 20 mm, and 12 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after the fourth layer excavation is at the upstream wall and downstream wall which is 88 mm as shown in Figure 21 below.

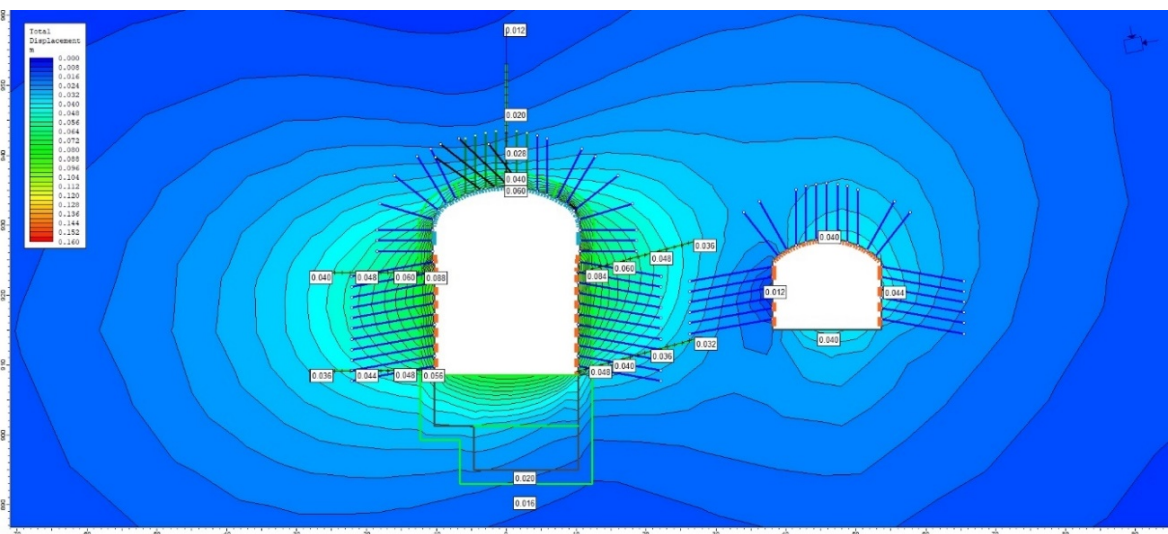


Figure 21 Displacement of monitoring points after fourth layer excavation

Likewise, the total deformation on the crown at Ch 0+39 after the fifth layer excavation and support is 63 mm, 41 mm, 27 mm, 18 mm, and 9 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after the fifth layer excavation is at the upstream wall and downstream wall which is 95 mm as shown in Figure 22 below.

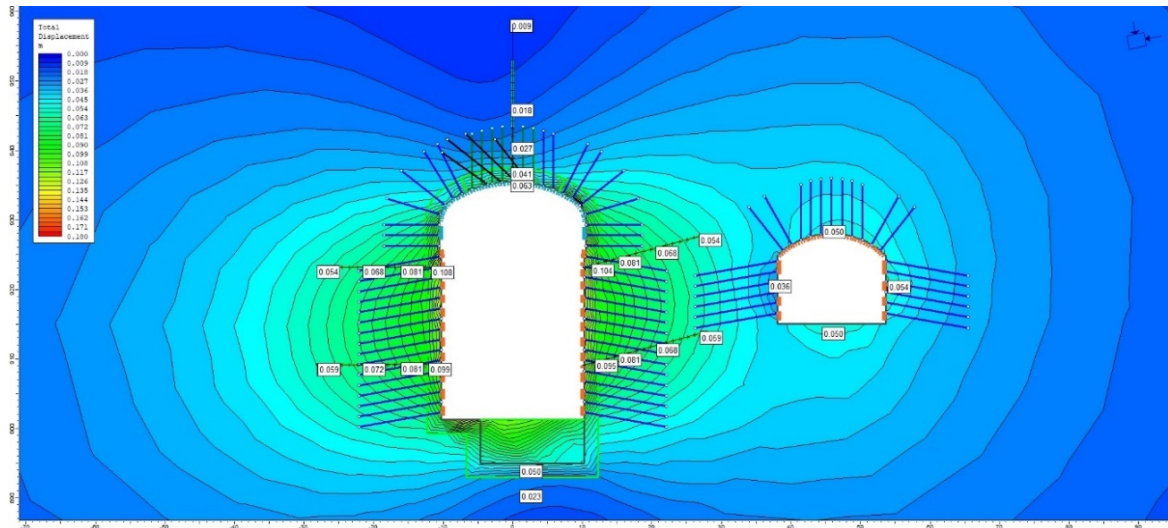


Figure 22 Displacement of monitoring points after fifth layer excavation

Finally, the total deformation on the crown at Ch 0+39 after the fifth layer excavation and support is 63 mm, 42 mm, 26 mm, 16 mm, and 10 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after the fifth layer excavation is at the upstream wall and downstream wall which is 115 mm as shown in Figure 23 below.

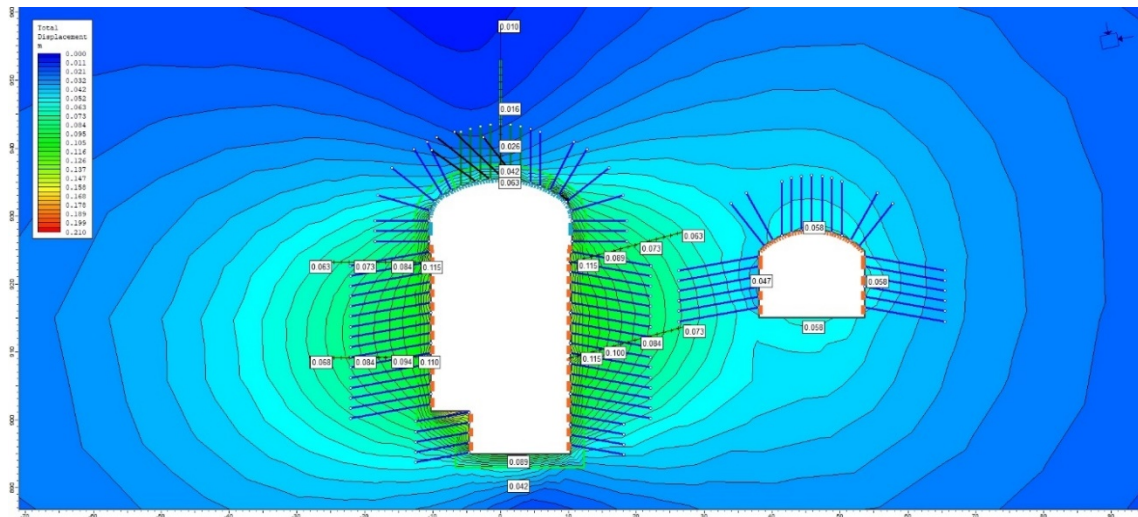


Figure 23 Displacement of monitoring points after final excavation

After the full excavation as shown in Figure 23, the maximum displacement distribution is centred at the upstream and downstream walls. The displacement distribution shows butterfly-type two lobes on each wall where the maximum displacement is observed near the wall boundary and gradually decreases far from the excavated surface. Ex1 is the extensometer installed at the crown at El.935.5 m, and Ex2 and Ex3 are the extensometer installed at the second layer at the upstream and downstream walls at elevation 923 m and similarly, Ex4 and Ex5 are also installed during the fourth excavation at elevation 909 m. The displacement versus distance prediction from the model on different extensometers points after first layer excavation and final excavation is shown in Figure 24, Figure 25, Figure 26, Figure 27, and Figure 28 below.

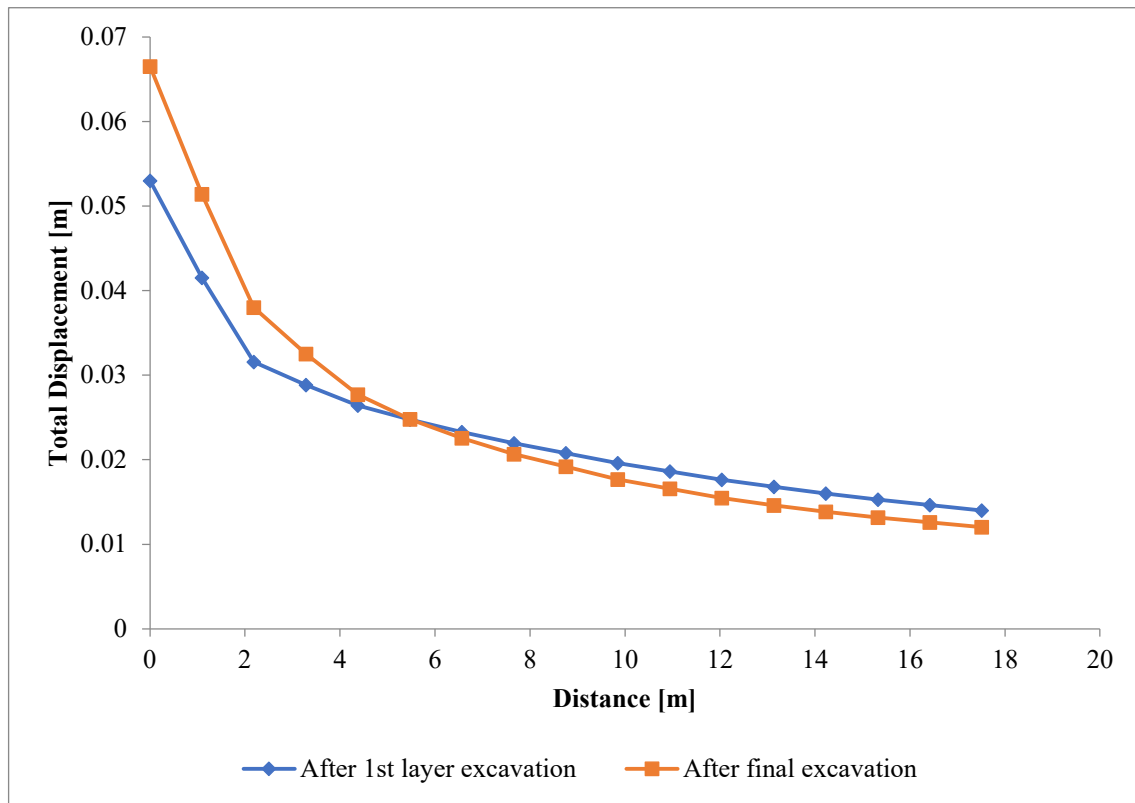


Figure 24 Total displacement on extensometer1 (Ex1)

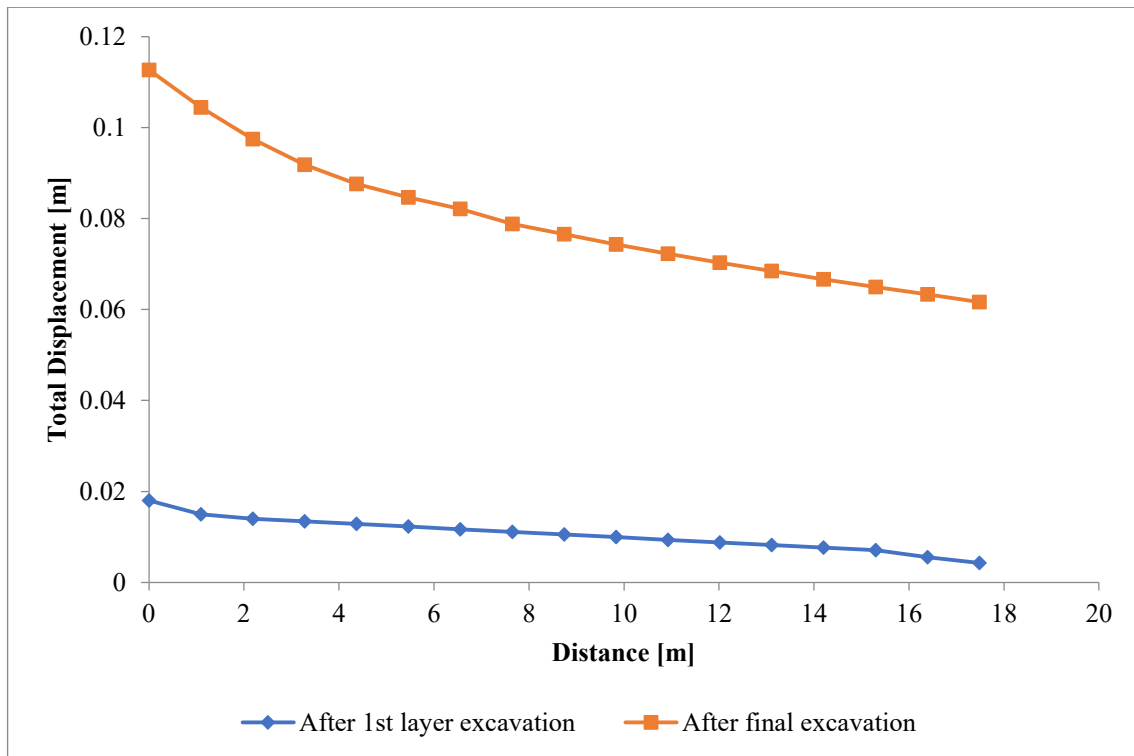


Figure 25 Total displacement on extensometer2 (Ex2)

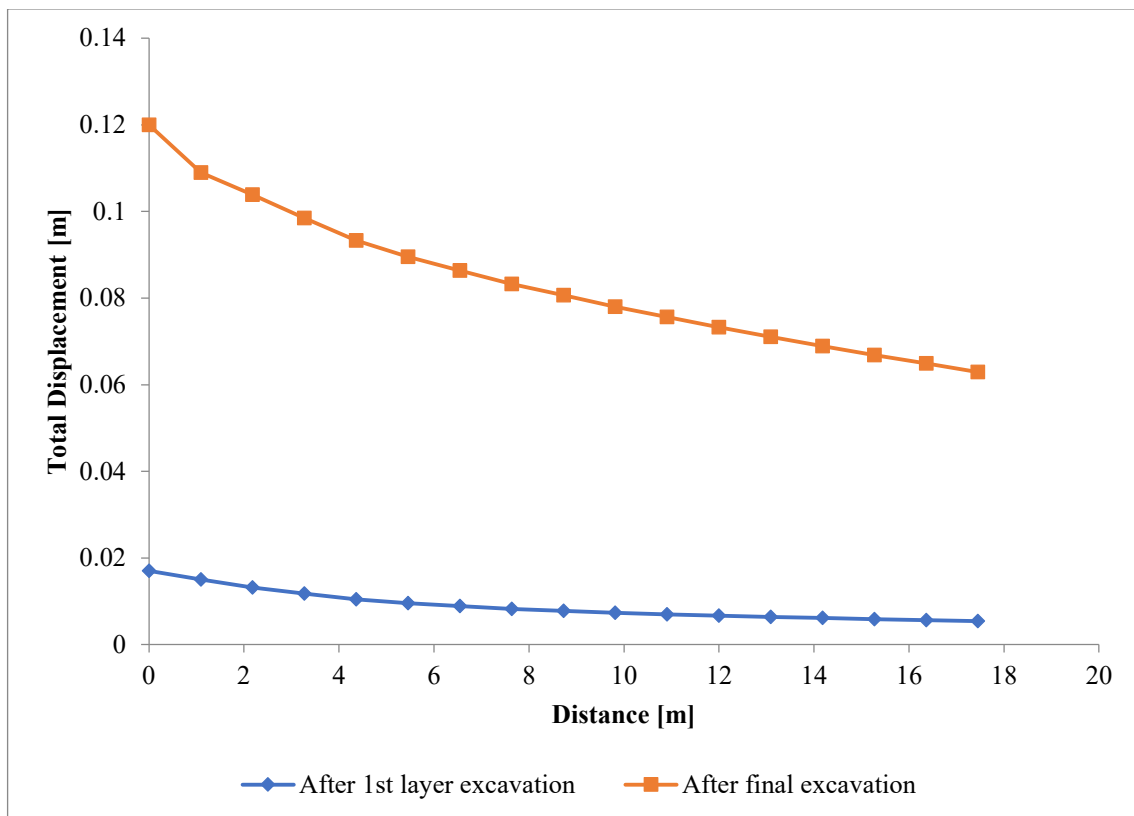


Figure 26 Total displacement on extensometer3 (Ex3)

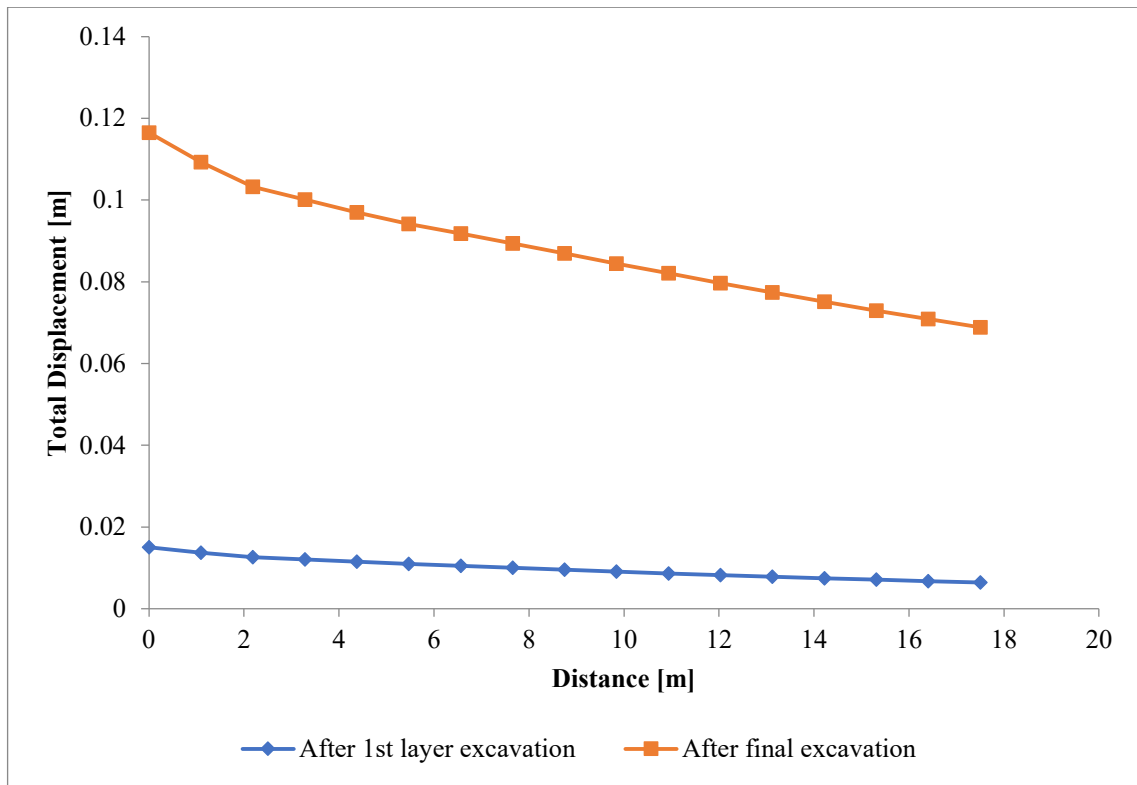


Figure 27 Total displacement on extensometer4 (Ex4)

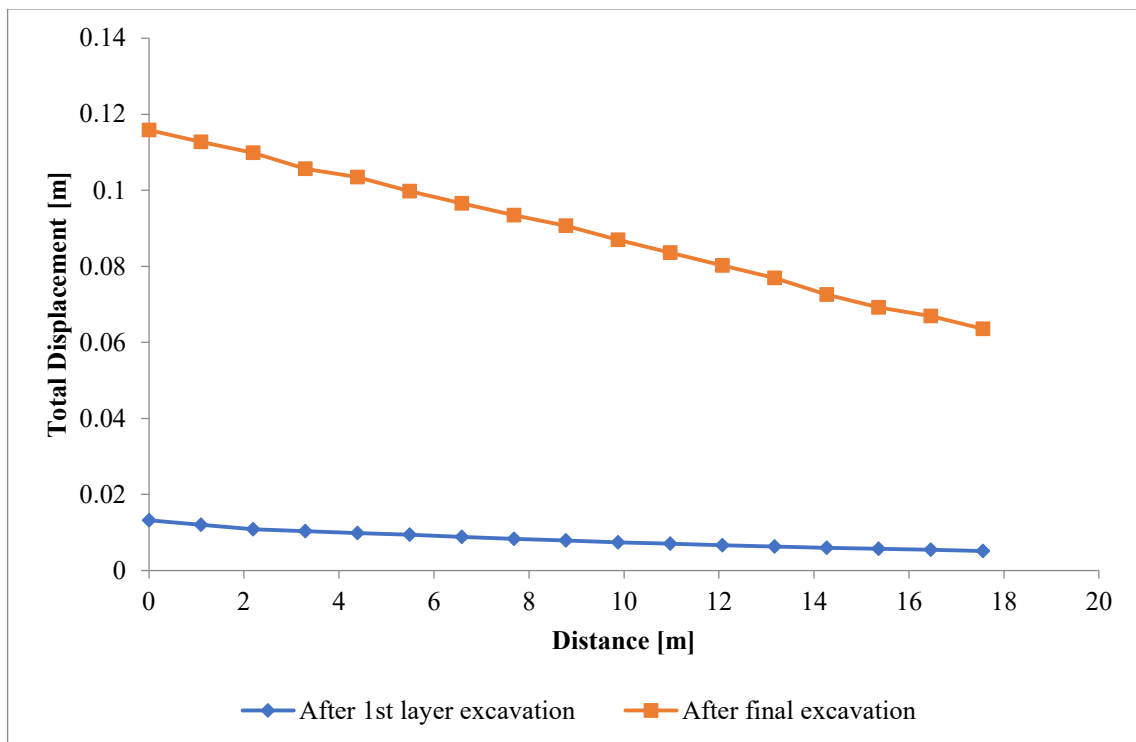


Figure 28 Total displacement on extensometer5 (Ex5)

4.3.3 Deformation monitoring analysis from the multipoint extensometer

In the Powerhouse cavern of Upper Trishuli-1, the displacement was measured through the six multipoint extensometers. The multipoint extensometer Ex1 was installed at the date of 2022.10.30 at Ch 0+39m. The deformation data is given in (Appendix K) measured by multipoint extensometer Ex1 and surface displacement data taken from convergence (Appendix M) was provided by Upper Trishuli-1. The deformation at this chainage is shown in figure 29 and the absolute and relative displacement measurements from the extensometer at point Ex1 after the first layer excavation are shown in Table 12.

Table 12 Total displacement of Ex1 after first layer excavation

Date:2023-2-28		Displacement at Ch 0+39m	
Extensometer			
Depth(m)	Absolute Displacement(mm)		Relative Displacement(mm)
0	52.5		-
2	7.27		34.77
5	27.93		21.65
10	42.41		7.17
17	49.58		2.92

the total relative deformation on the crown at Ch 0+39 from the extensometer after the first layer excavation and support is 49.58 mm, 34.77 mm, 21.65 mm, 7.17 mm, and 2.92 mm at 0m, 2m, 5m, 10m, and 17m respectively. From the convergence data the surface displacement shows 52.5 mm.

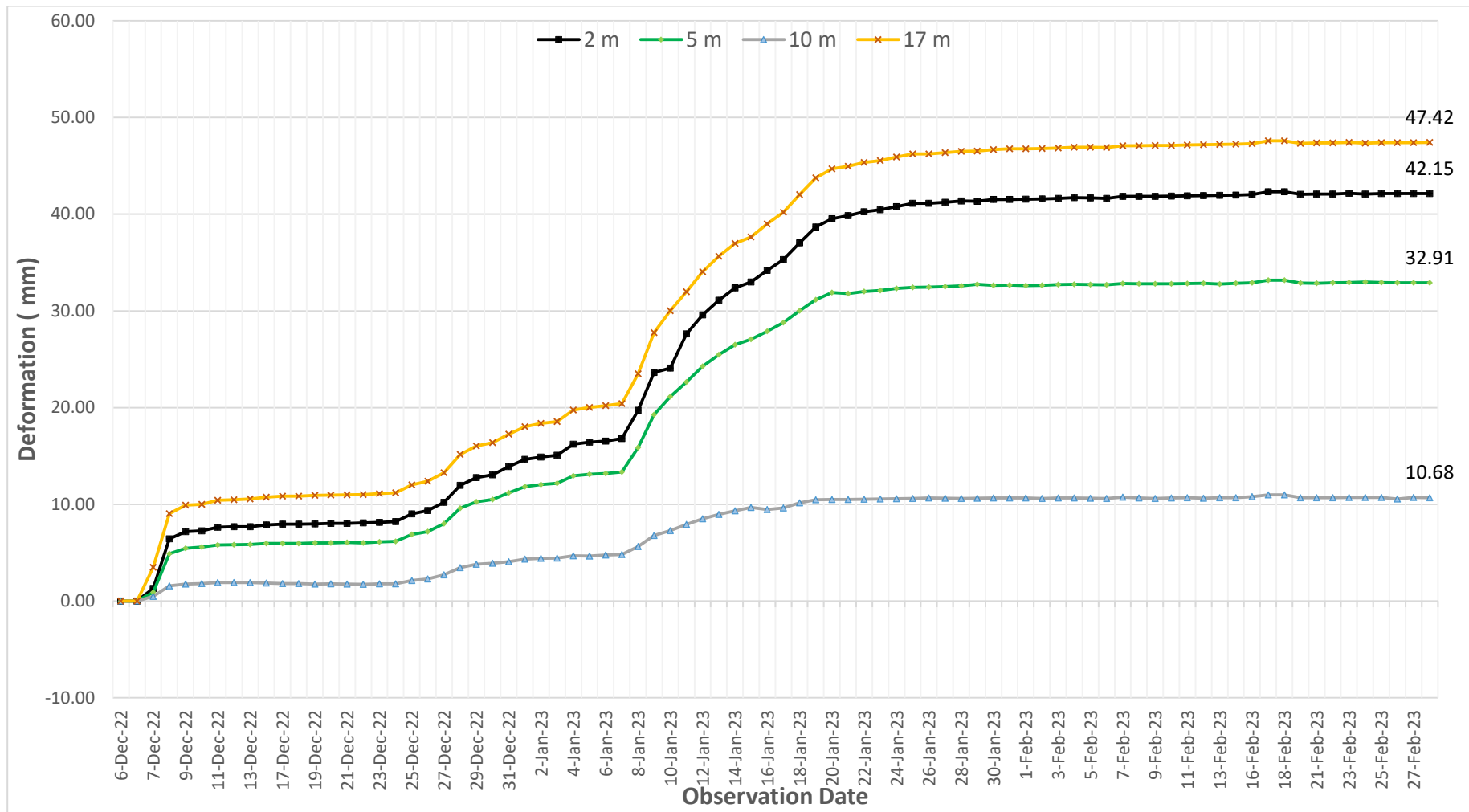


Figure 29 Total displacement measured from the extensometer at Ch 0+39

4.3.4 Stress distribution after excavation and support at Ch 0+67 m

The stress distribution at chainage 0+67 of the powerhouse cavern was performed using the Phase2 software which is based on the finite element method. The major principal stress σ_1 distribution at crown along the extensometer points Ex1 is given in Figure 331. The value of σ_1 is constant at in-situ conditions and gradually changes with the excavation sequences as shown in Figure 30. At the first layer of excavation, σ_1 varies between 3.5 MPa to 10.76 MPa and gradually increases to the final layer of excavation. The σ_1 distribution is maximum at the crown and lower part of the invert and minimum at the side wall and ranges from 1.65MPa to 13.75MPa after the final layer excavation.

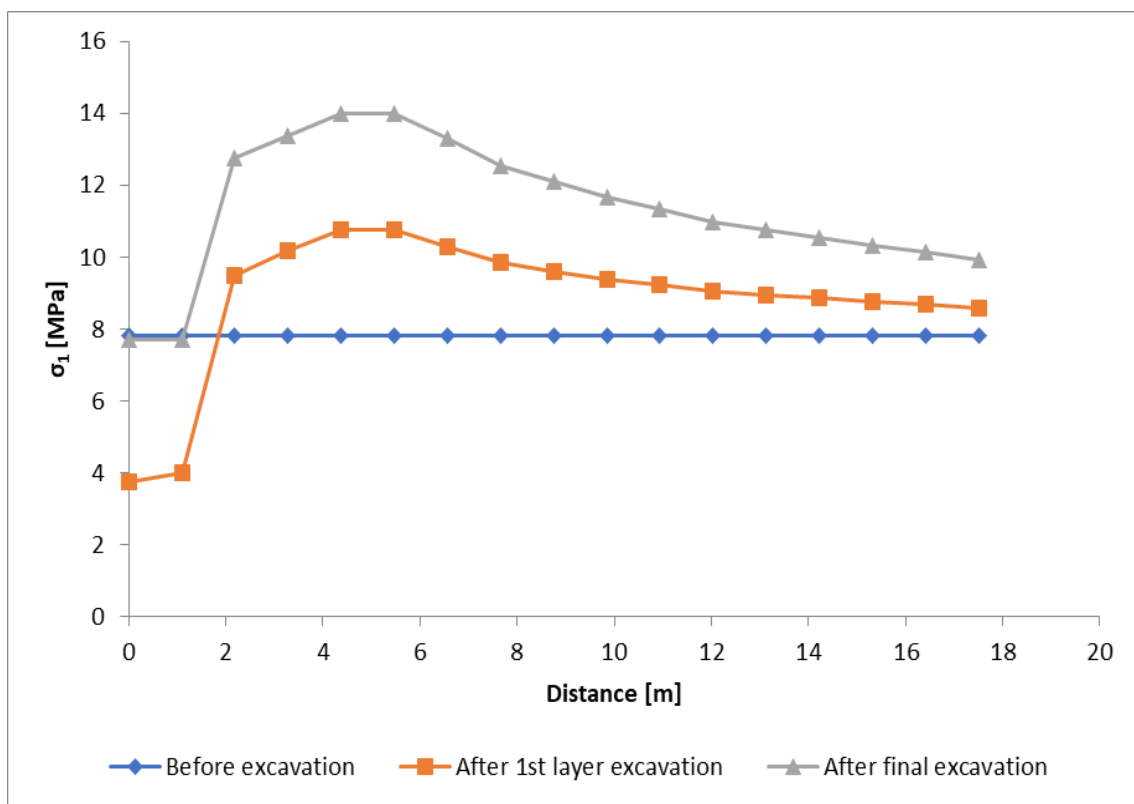


Figure 30 Distribution of sigma 1 stress magnitude at chainage 0+67m before excavation, first layer, and final layer excavation

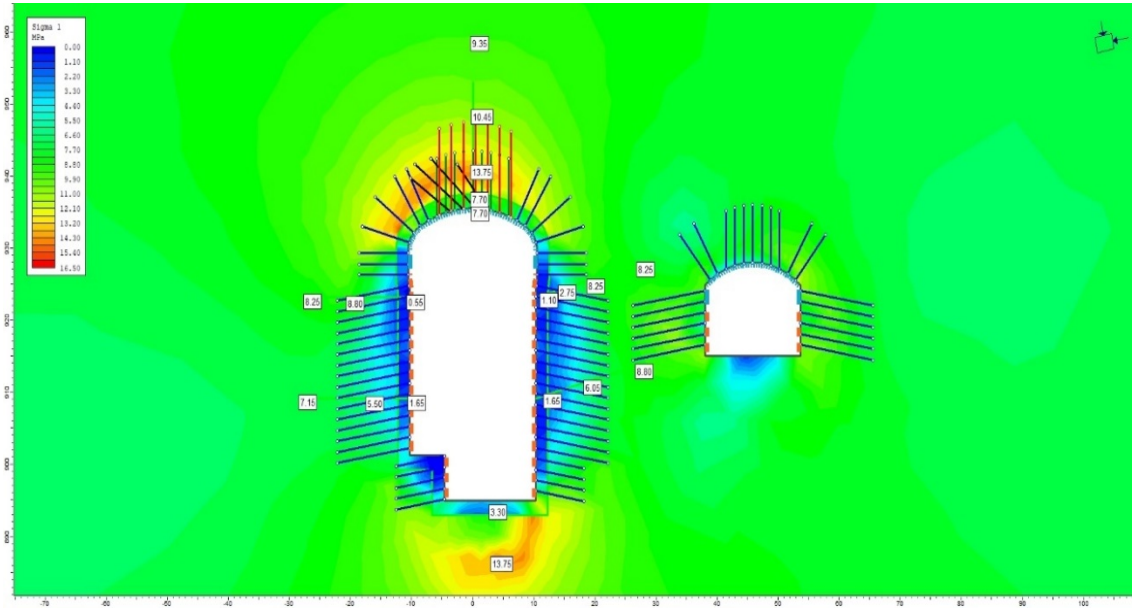


Figure 31 Distribution of sigma 1 stress magnitude at chainage 0+67 m

Also, the distribution of σ_3 ranges from 0.25 MPa to 5.27 MPa as shown in Figure 33. The distribution of the magnitude decreased along the excavation line after the complete excavation. The distribution of the magnitude of σ_3 stress at chainage 0+67 m before excavation, during, and after excavation is shown in Figure 32 below.

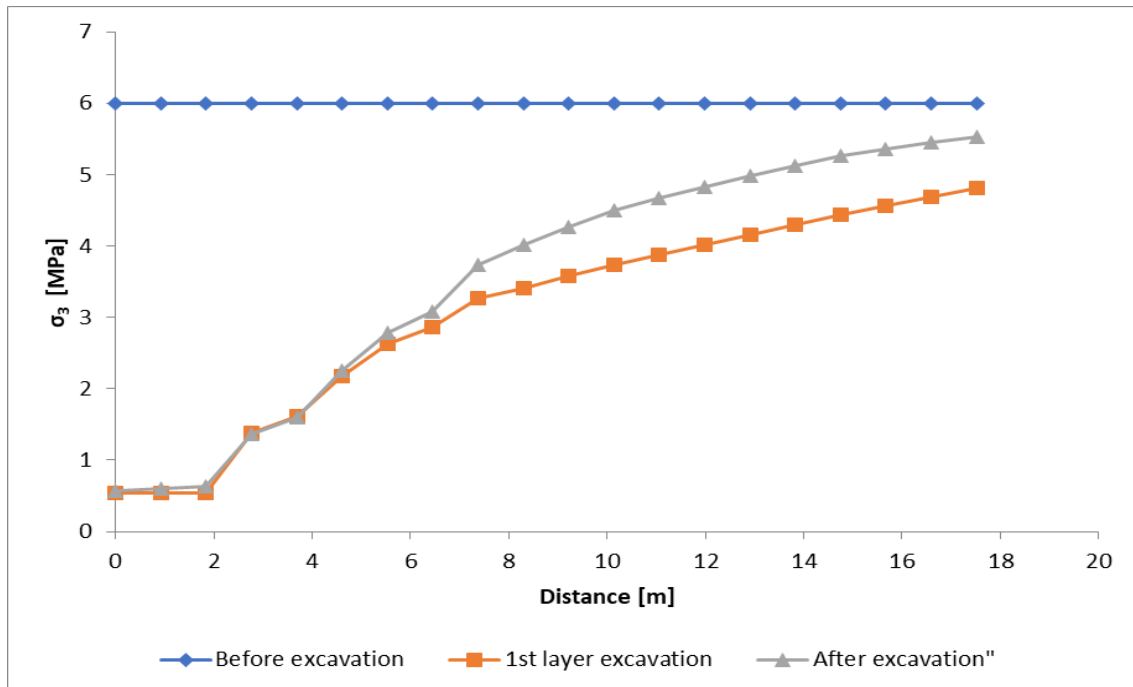


Figure 32 Distribution of sigma 3 stress magnitude at chainage 0+39m before excavation, first layer, and final layer excavation

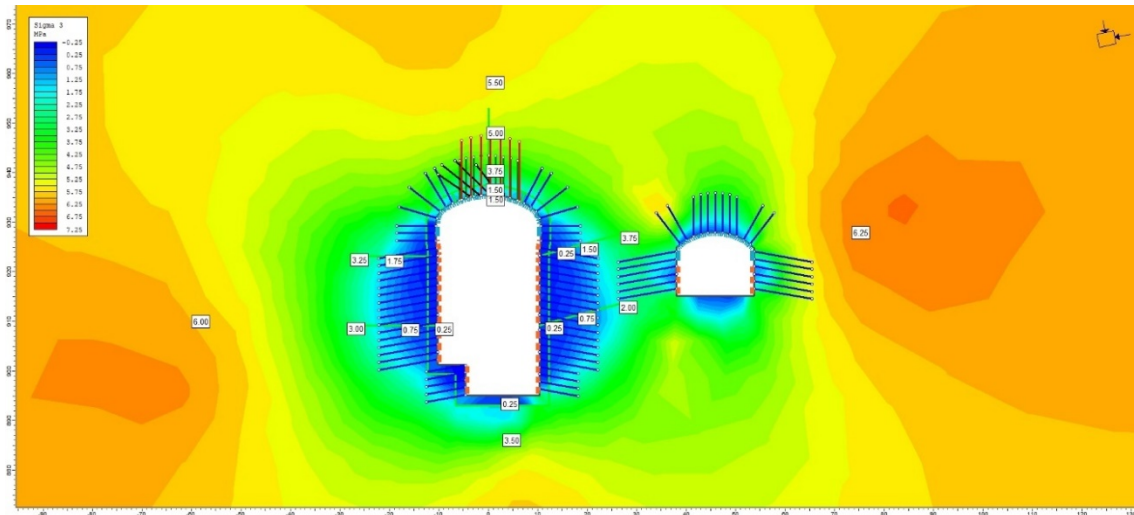


Figure 33 Distribution of sigma 3 stress magnitude at chainage 0+67 m

Similarly, the distribution of mean stress ranges from 0.9 MPa to 7.85 MPa as shown in Figure 35. The distribution of the magnitude of mean stress decreased along the excavation line after the complete excavation. The mean stress increased at the top and bottom of the excavated surface. The distribution of the magnitude of mean stress at chainage 0+67m before excavation, during, and after excavation is shown in Figure 34 below.

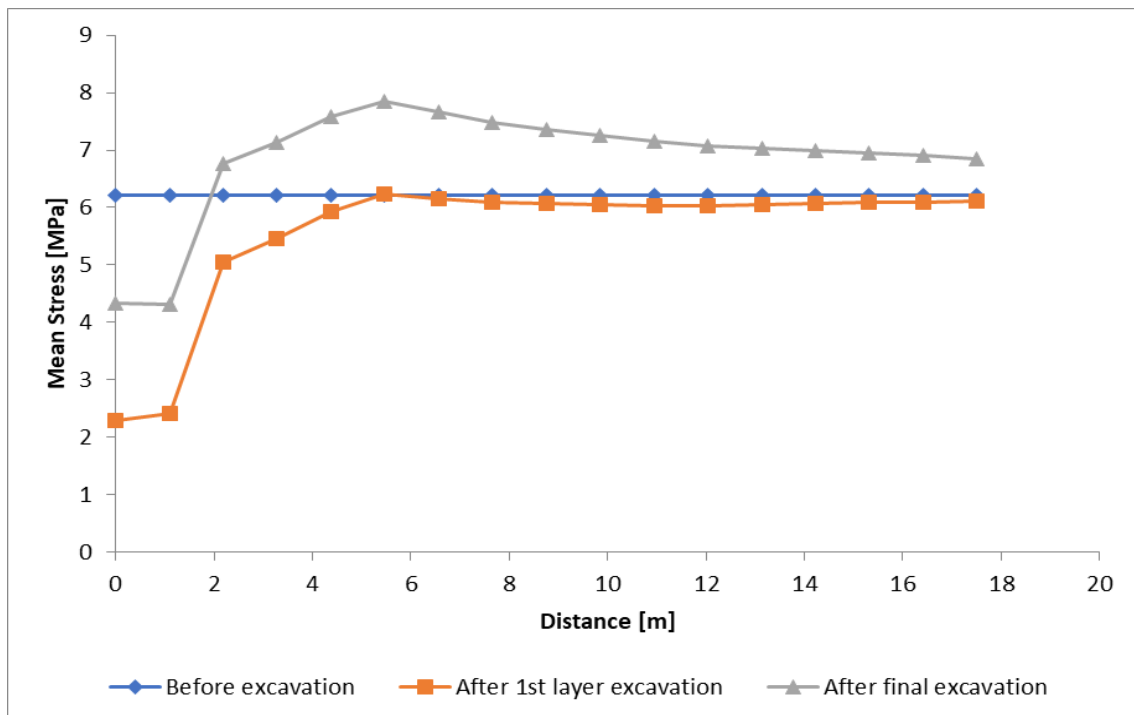


Figure 34 Distribution of mean stress magnitude at chainage 0+67m before excavation, first layer, and final layer excavation

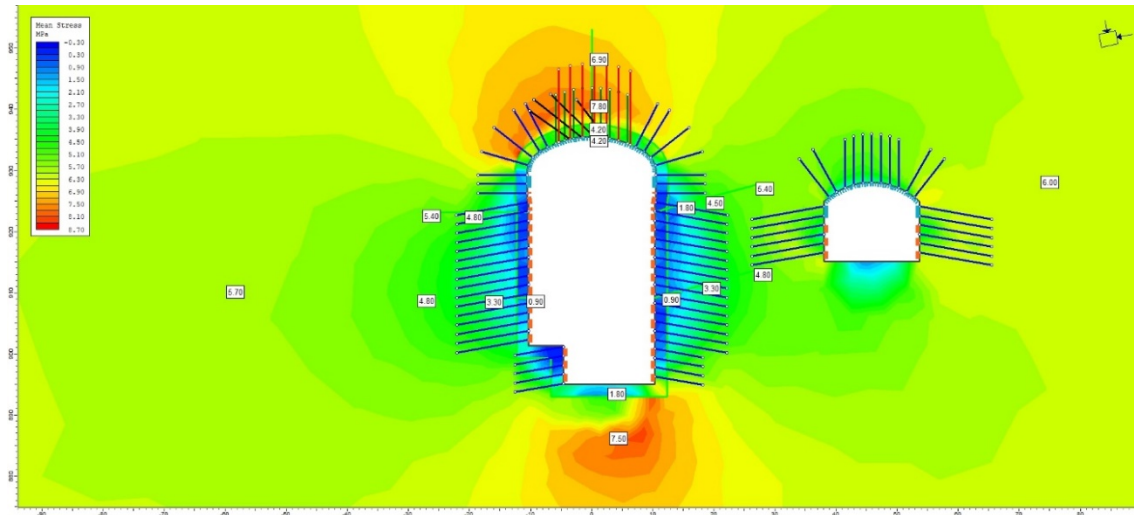


Figure 35 Distribution of mean stress magnitude at chainage 0+67 m

4.3.5 Deformation of Powerhouse Cavern at Ch 0+67 m

According to the FEM calculation result, the displacement of monitoring points (Appendix F) at each excavation stage is shown in Figure 36. The total relative deformation on the crown at Ch 0+67 after the first excavation and support along the crown is maximum than at side walls. The displacement at the crown is 54 mm, 42 mm, 30 mm, 18 mm, and 12 mm at 0m, 2m, 5m, 10m, and 17m respectively. The maximum displacement at Ex1 after the first layer excavation from the model is 54 mm and the minimum is 9 mm.

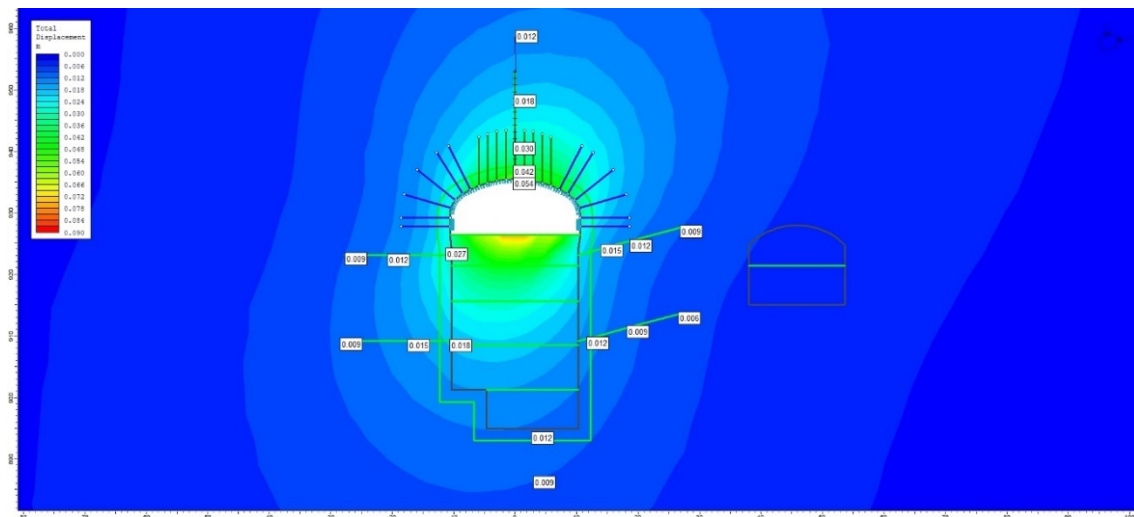


Figure 36 Displacement of monitoring points Ex1 after first layer excavation

From the model, the total deformation on the crown at Ch 0+67 after the second layer excavation and support is 57 mm, 42 mm, 30 mm, 21 mm, and 15 mm at 0m, 2m, 5m, 10m,

and 17m respectively. The crown and bottom have a maximum deformation after the second layer excavation which is 57 mm as shown in Figure 37 below. During this excavation, the transformer cavern's first layer was also excavated simultaneously.

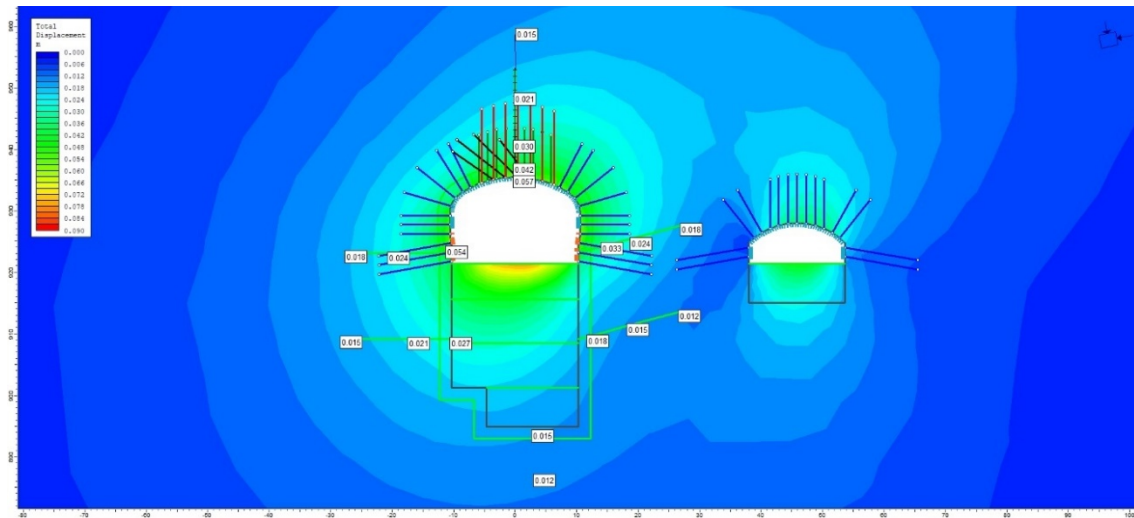


Figure 37 Displacement of monitoring points Ex1 after second layer excavation

Similarly, the total deformation on the crown at Ch 0+67 from the model after the third layer excavation and support is 60 mm, 45 mm, 33 mm, 24 mm, and 18 mm at 0 m, 2 m, 5m, 10 m, and 17 m respectively. The maximum deformation after third layer excavation is at the downstream wall and downstream wall which is 81 mm as shown in Figure 38 below. The second layer of the transformer cavern was excavated simultaneously.

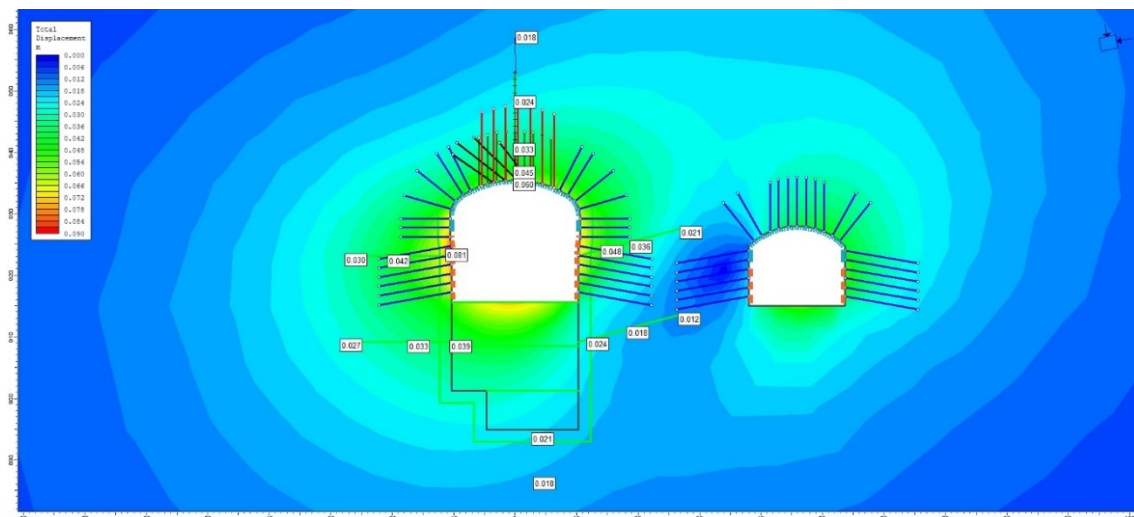
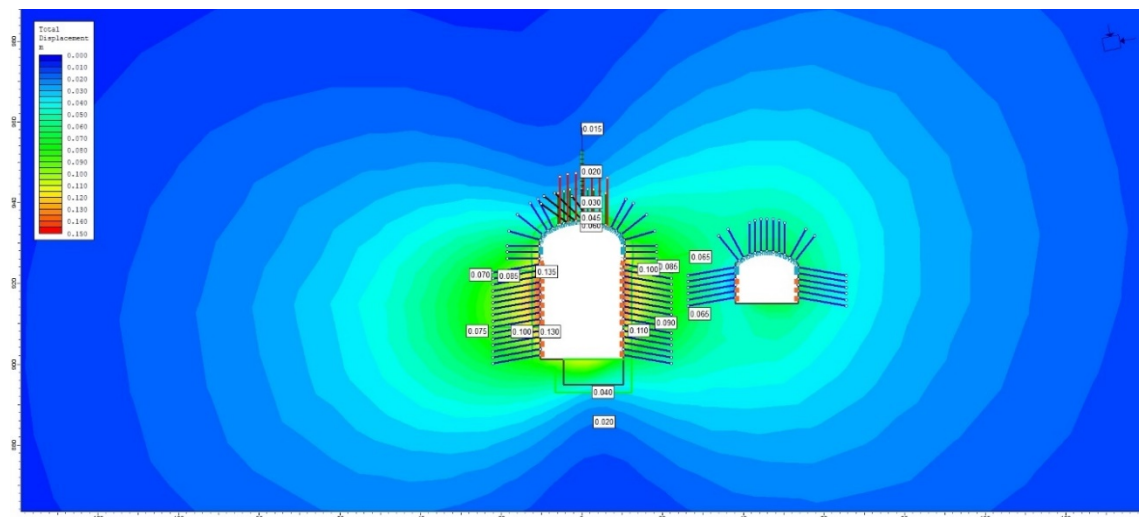


Figure 38 Displacement of monitoring points after third layer excavation

Likewise, the total deformation on the crown at Ch 0+67 after the fifth layer excavation and support is 60 mm, 45 mm, 30 mm, 20 mm, and 15 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after the fifth layer excavation is at the upstream wall and downstream wall which is 135 mm as shown in Figure 40 below



61

Finally, the total deformation on the crown at Ch 0+67 after the fifth layer excavation and support is 60 mm, 44 mm, 33 mm, 22 mm, and 17 mm at 0 m, 2 m, 5 m, 10 m, and 17 m respectively. The maximum deformation after the fifth layer excavation is at the upstream wall and downstream wall which is 155 mm as shown in Figure 41 below

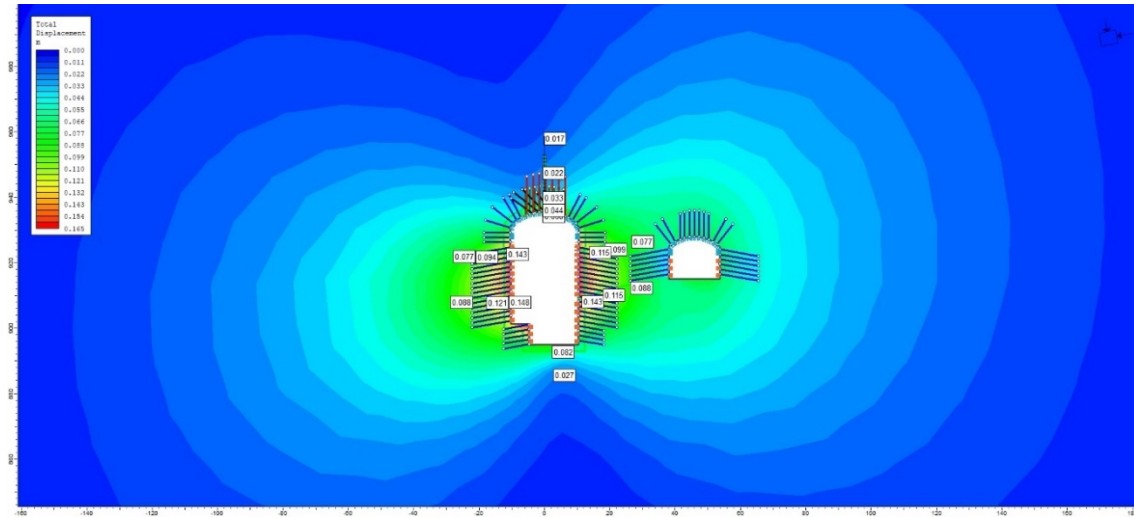


Figure 41 Displacement of monitoring points after final excavation

After the full excavation as shown in Figure 41, the maximum displacement distribution is centred at the upstream and downstream walls. The displacement distribution shows butterfly-type two lobes on each wall where the maximum displacement is observed near the wall boundary and gradually decreases far from the excavated surface. Ex1 is the extensometer installed at the crown at El.935.5 m, and Ex2 and Ex3 are the extensometer installed at the second layer at the upstream and downstream walls at elevation 923m and similarly, Ex4 and Ex5 are also installed during the fourth excavation at elevation 909m. The displacement versus distance on different extensometers after first layer excavation and final excavation is shown in Figure 42, Figure 43, Figure 44 Figure 45, and Figure 46 below.

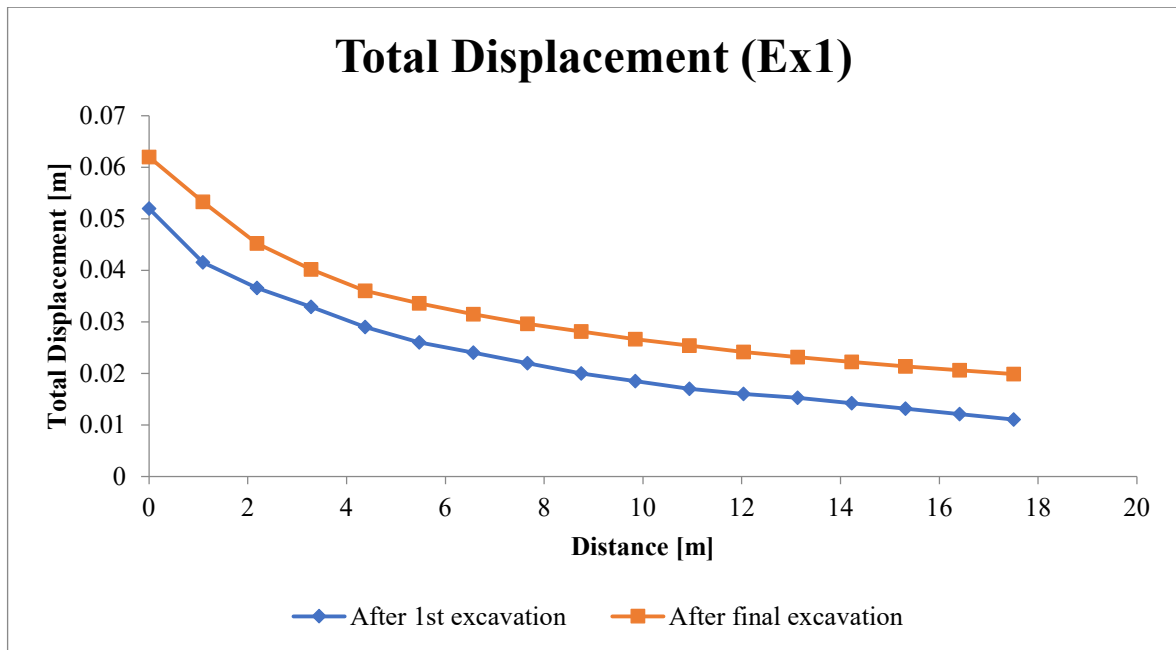


Figure 42 Total displacement on extensometer1 (Ex1)

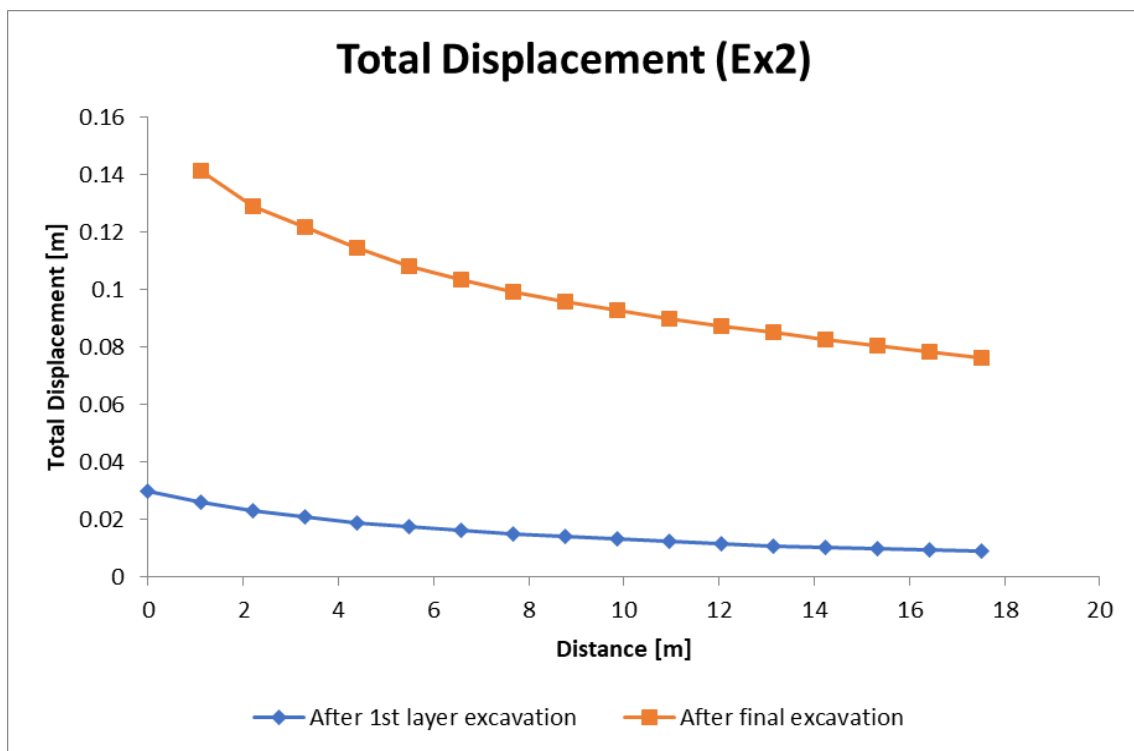


Figure 43 Total displacement on extensometer2 (Ex2)

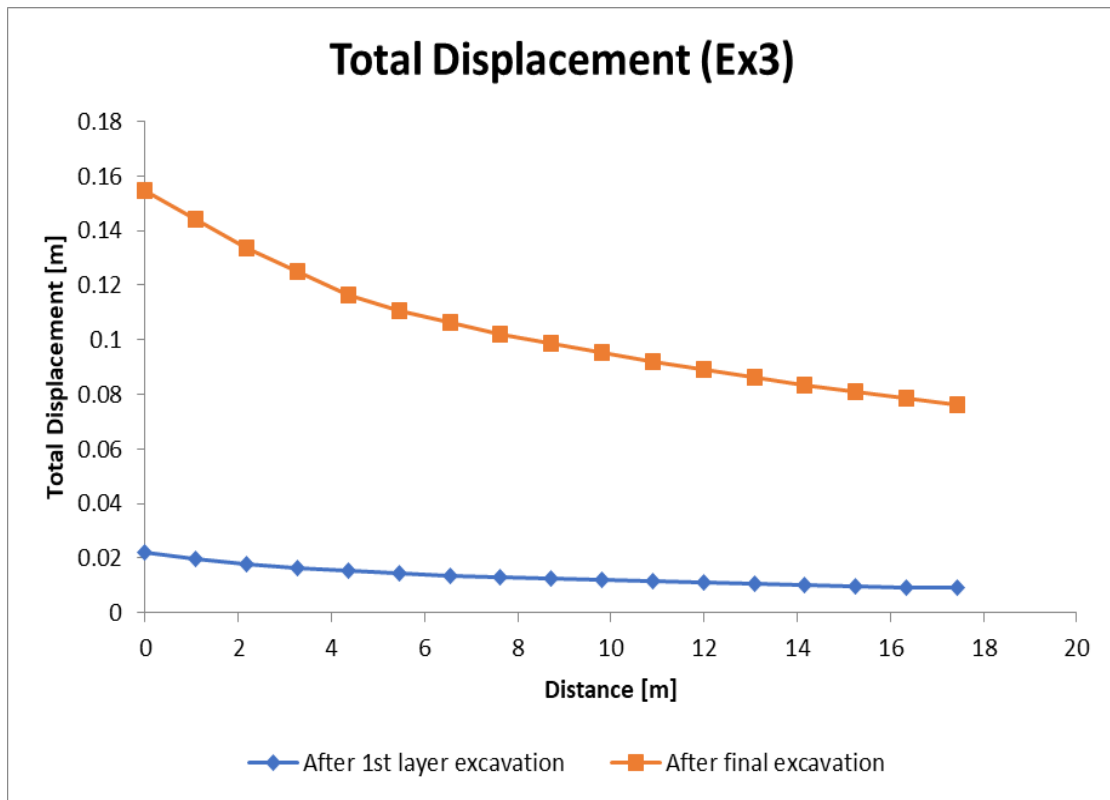


Figure 44 Total displacement on extensometer3 (Ex3)

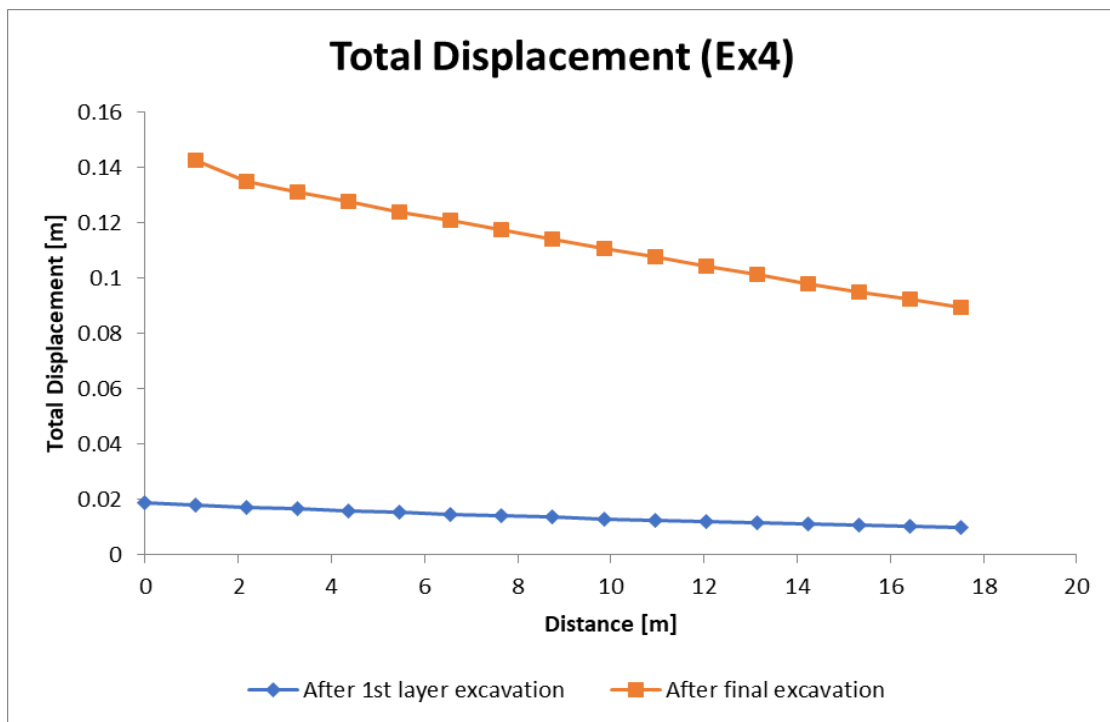


Figure 45 Total displacement on extensometer4 (Ex4)

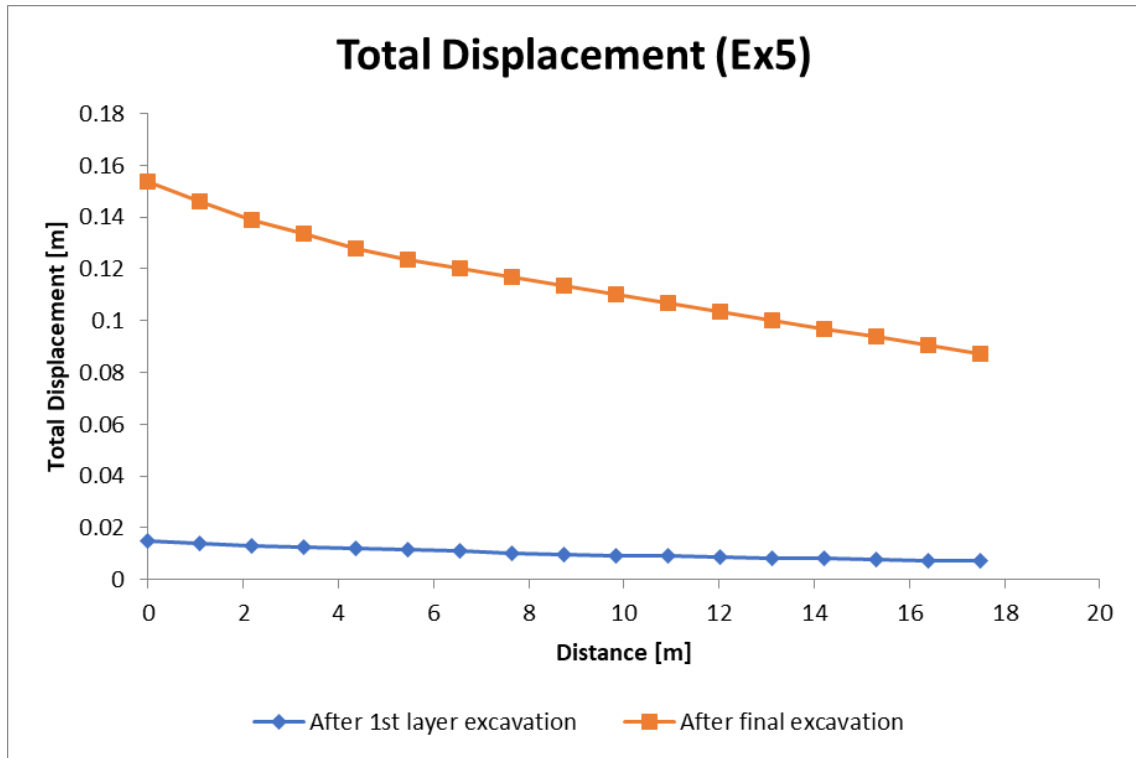


Figure 46 Total displacement on extensometer5 (Ex5)

4.3.6 Deformation Monitoring Analysis from multipoint extensometer at Ch 0+67 m

In the Powerhouse cavern of Upper Trishuli-1, the displacement was measured through the six multipoint extensometers. The multipoint extensometer Ex1 was installed at the date of 2022.12.06 at Ch 0+ 67. The deformation data (Appendix L) measured by multipoint extensometer Ex1 was provided by Upper Trishuli-1. The relative displacement measured at Powerhouse cavern from the multipoint extensometer is shown in Figure 47.

Table 13 Total displacement measured from extensometer and predicted from the model after first layer excavation

Displacement at Ch 0+67 m			Date:2023-2-28	
Extensometer			Model	
Depth(m)	Absolute Displacement(mm)	Relative Displacement(mm)	Depth(m)	Total Displacement (mm)
0	54.8	-	0	54.66
2	5.27	42.15	2	42.42
5	14.51	32.91	5	30.20
10	36.73	10.68	10	18.74
17	47.42	7.38	17	12.08

4.3 Comparative studies

The comparative studies were made with the overall geological condition and the deformation of powerhouse cavern. The validation of numerical model is made by comparing with the observed monitoring data.

Geological and engineering geological studies

In terms of engineering geological conditions, the powerhouse cavern in the study area is mainly composed of psammitic and pelitic schists with four major joint sets with tight to wide aperture. This is similar to other rock masses in the Lesser Himalaya, which are characterized by closely spaced joint sets with varying orientations and persistence. Based on the geological and engineering geological studies, the powerhouse cavern roof consists of a disturbed rock mass with an uneven distribution of quartz veins mostly associated with pelitic schist. Additionally, a clay band was observed at Ch 0+67m at a depth of 6.7m. A weak bands and quartz veins are localized to the area between 5m to 10m depth. The hemispherical plot from the combination of joints shows the area with prismatic and tetrahedral wedge formed at the crown.

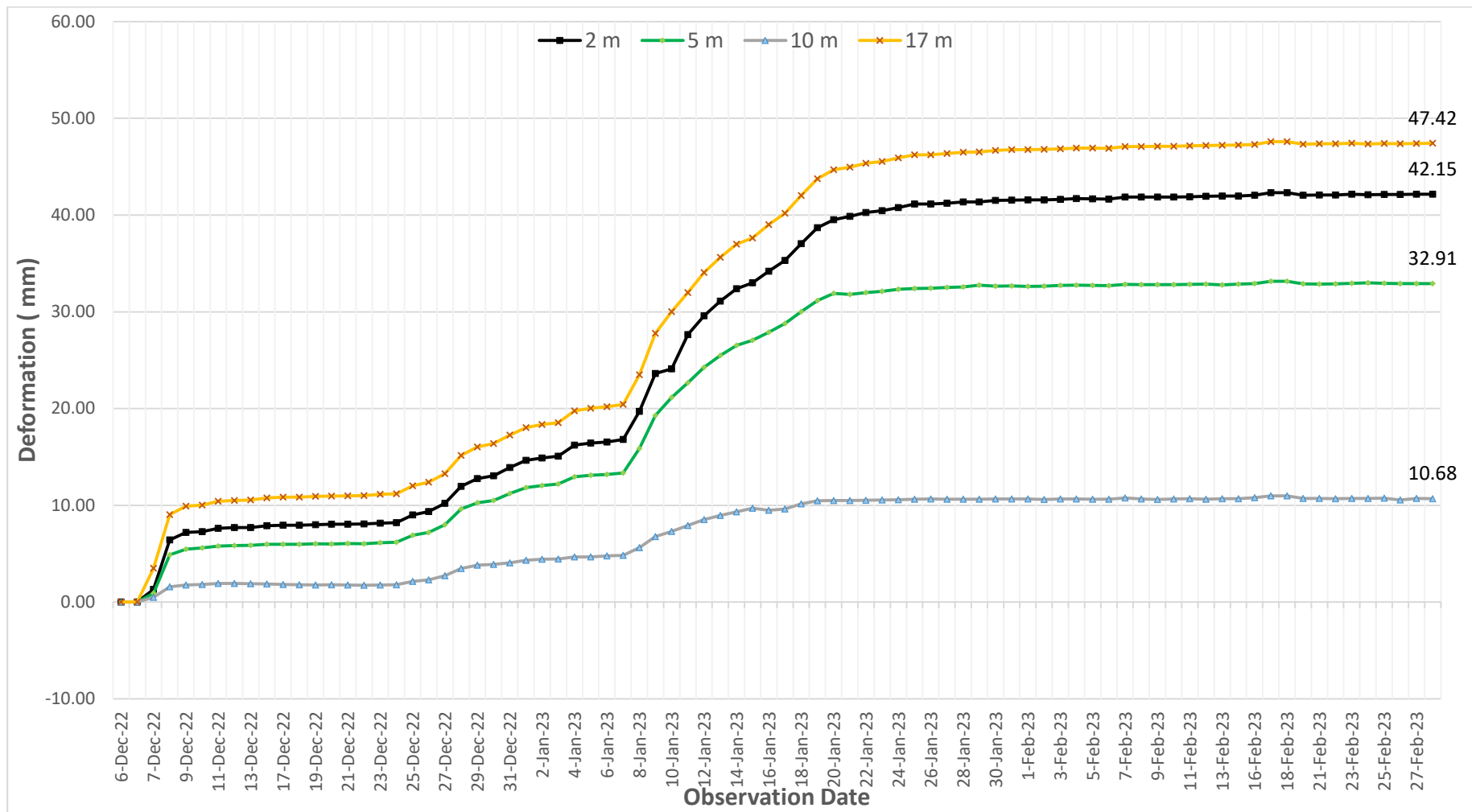


Figure 47 Deformation measurement of the powerhouse cavern roof from Ex1 at Ch 0+67m

Geotechnical properties of rock mass

This section describes the geomechanical classification of rock mass in the underground powerhouse cavern using various rock mass classification systems, including RQD, RMR, Q, and GSI. Rock mass classification were done in the field after every excavation cycle and summarized after the completion of the first layer. The values were calculated at different intervals of 5m along the cavern's alignment. The RMR value ranges from 46 to 57, The RQD value ranges from 52 to 72, the GSI value ranges from 39 to 52 and the Q-value ranges from 4.15 to 7.21. The distribution of RMR, GSI, Q, and RMR from the Q system shows small changes in rock mass where RMR from Q estimates the maximum, and GSI and RMR give closer values. The overall rock mass classification shows the rock mass of the powerhouse cavern as fair rock.

From the laboratory test the UCS of rock material ranges from 38.96 to 81.03 MPa with an average UCS of 60.85 MPa. Similarly, the rock mass strength parameters estimated using Howk- Brown failure criteria show the average mb value ranges from 1.225 to 0.771, s value ranges from 0.007 to 0.0027 and a value range from 0.506 to 0.513. The deformation modulus of rock mass was estimated using (Hoek and Diederichs, 2006), and the value ranges form 3.98 to 1.92 with an average E_m 2.95 MPa.

Numerical model and extensometer

A comparative study was made between the observed and predicted deformation. At first the comparative study is made for chainage 0+39m. The maximum displacement measured after the first layer excavation from the extensometer Ex1 is 49.58 mm and from the model is 55.05 mm as shown in table 14 below. Similarly, the minimum displacement from the extensometer is 1.81 mm and from the model is 11 mm. From the convergence data (Appendix L), The surface displacement on the crown is 52.5 mm. The relative displacement curve from extensometer data follows the equation $y=50.97e^{-0.183x}$ where x is the depth of the extensometer with trendline reliability ($R^2=0.9915$). Similarly, the displacement from the model follows the equation $y=43.788e^{-0.081x}$ with trendline reliability ($R^2=0.9379$) as shown in Figure 48. The extensometer and model both shows the similar displacement upto 5m anchor depth and above this anchor the model overestimates the displacement value. Similarly, the displacement on anchor between depth of 0m to 2m is 14.81 mm, 2m 5m is

13.12 mm and 5, to 10 m shows 14.48 mm deformation. Here, the anchor between 5 to 10m depth shows the maximum deformation then the anchor between 0m, 2m and 5m depth.

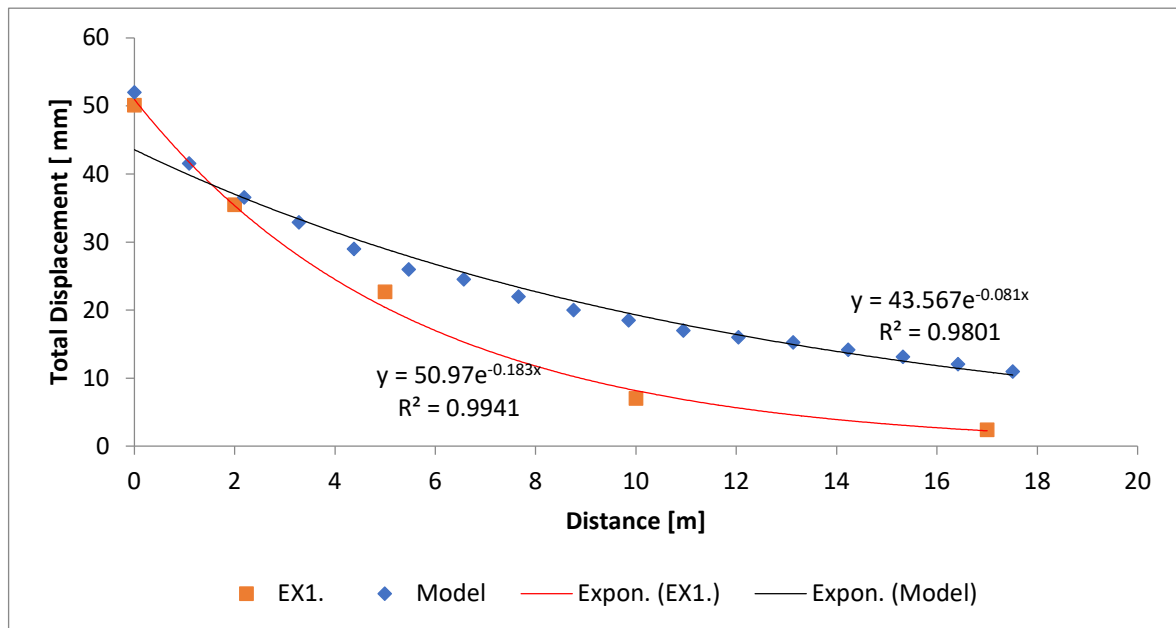


Figure 48 Total displacement curve of measured and predicted data (Ch 0+39 m) after 1st layer excavation

Table 14 Total displacement measured from extensometer and predicted from model after first layer excavation

Displacement at Ch 0+39m			Date:2023-2-28	
Extensometer			Model	
Depth(m)	Absolute Displacement (mm)	Relative Displacement (mm)	Depth(m)	Total Displacement(mm)
0	52.5	-	0	52.9
2	7.27	34.77	2	36.58
5	27.93	21.65	5	24.76
10	42.41	7.17	10	19.62
17	49.58	2.92	17	11.00

Similarly, the comparative study was made for chainage 0+67m. The absolute and relative displacement measurements from the model and extensometer at point Ex1 after the first layer excavation is summarized in table 15. The surface displacement at Ch 0+63m have 54.8 mm displacement from the convergence data given in Appendix N

The maximum displacement measured after the first layer excavation from the extensometer is 47.42 mm and from the model is 55.05 mm. Similarly, the minimum displacement from the extensometer is 7.38 mm and from the model is 12.08 mm. The relative displacement curve from extensometer data follows the equation $y=53.348e^{-0.126x}$ where x is the depth of the extensometer with trendline reliability ($R^2=0.9773$). Similarly, the displacement from the model follows the equation $y=49.625e^{-0.084x}$ with trendline reliability ($R^2=0.9773$) as shown in Figure 49.

Table 15 Total displacement measured from extensometer and predicted from the model after first layer excavation

Displacement at Ch 0+67 m			Date:2023-2-28	
Extensometer			Model	
Depth(m)	Absolute Displacement(mm)	Relative Displacement(mm)	Depth(m)	Total Displacement (mm)
0	54.8	-	0	54.66
2	5.27	42.15	2	42.42
5	14.51	32.91	5	30.20
10	36.73	10.68	10	18.74
17	47.42	7.38	17	12.08

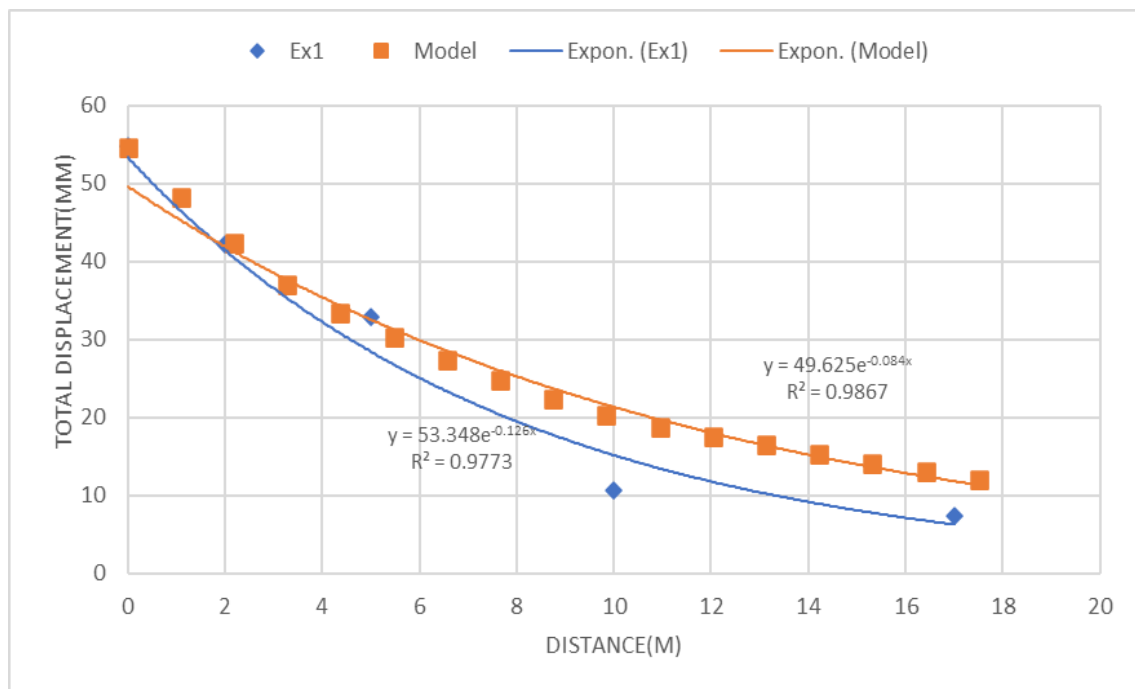


Figure 49 Total displacement curve of measured and predicted data (Ch 0+67 m) after 1st layer excavation

The extensometer and model both shows the similar displacement upto 5m anchor depth and above this anchor the model overestimates the displacement value. Similarly, the displacement on anchor between depth of 0m to 2m is 12.65 mm, 2m 5m is 9.24 mm and 5, to 10m shows 22.23 mm deformation. Here, the anchor between 5 to 10m depth shows the maximum deformation then the anchor between 0m, 2m and 5m depth.

From the geological study of powerhouse cavern using probe-hole log, the roof of powerhouse cavern above 5m at Ch 0+37.5m consists of uneven distribution of quartz veins which is mostly associated with weak band of pelitic Schist. Some open joints are also along the depth between 5 to 10m. Similarly at Ch 0+67m, A small clay band associated with the weak rock was observed at the depth of 6.7m and the area between 5m to 10m depth have numerous weak bands and quartz vein associated with it. The numerical model shows the deformation gradually decreases with depth is due to the method that assumes the material and structure being analysed behaves linearly under the applied loads.

CHAPTER V

DISCUSSION

To achieve the research work, geological and geotechnical studies including the rock mass classification were conducted, and the analysis related to stress distribution and deformation of the powerhouse cavern was performed. The obtained results from the field and research work are discussed below:

5.1 Geology of study area

The geology of the study area was made using the topographic map of SOMDAN on a scale of 1:50000. Stocklin and Bhattarai (1997; 1980), divided the rock of Central Nepal into Nawakot complex and the Kathmandu complex separated by Mahabharat Thrust. KC and Paudyal (2019) made a similar study along the Ramche-Chhyamthali area and the Ulleri augen gneiss have been mapped at the confluence of the Trishuli and Mailung Khola of the Rasuwa district surrounded by the Kuncha Formation. The study made an observation that the Ulleri Augen Gneiss is exposed within the schist, phyllite, and metasandstone of the Kuncha Formation of the Lesser Himalaya. The present study obtains the similarity to them and classification and the units are named accordingly. The engineering geological condition of underground powerhouse cavern shows the Psammitic schist and Pelitic schist with major joints set with few random joints. These joints combined together form a prismatic wedge and tetrahedral wedges along the roof and side wall of the powerhouse cavern. The seepage combined with the weak rock mass (pelitic schist) creates a secondary clay mineral with swelling due to weathering.

5.2 Geotechnical properties of rock mass

5.2.1 Estimation of rock mass quality

The rock mass classification was estimated based on the Bieniawski (1989), Q-system updated by Grimstand and Barton (1993), and GSI updated by Hoek and Brown and further modified by Sonmez and Ulusay (1999). From the calculation, all rock mass comes under rock class III which is a fair rock in the powerhouse cavern. After the comparison of the distribution of different rock mass classification systems, the result shows similar rock classes but the value varies while comparing with the empirical relations. The values of rock

mass classification may vary within the same section at cavern due to the larger width of section and different properties of rock mass associated with it.

5.2.2 Rock mass strength

The strength of intact rock was estimated at the laboratory which shows an average value of 60MPa. The relationship provided by Hoek et al. (2002) Rock mass strength was estimated using the equation 7,8,9 and 10. The estimation was based on the GSI value from (Table 9). The parameter m_i was taken from the standard chart in Appendix D. Table 12 gives the estimated rock mass strength parameters.

5.2.3. Rock mass deformability

Rock mass deformability is needed for the numerical analysis in Phase2. An empirical relations based on GSI were used for deformation analysis among which the average value of 2.1 to 14.9 GPa. The difference in such a large range is due to the sensitivity of mathematical equations and the mode of function (Panthee et al. 2016). Hoek and Diederichs (2006) gives a relation based on GSI that provides a reliable estimate when intact rock modulus or intact rock strength are available. The intact rock modulus data was provided by Upper Trishuli-1 and intact rock strength test were done in laboratory so, the relation given by Hoek and Diederichs (2006) was used for numerical simulation that provides the similar deformation to the extensometer data observed at powerhouse cavern.

The disturbance due to blasting is more at 2m section, so 0.5 is take as disturbance factor which estimates less E_m for 2 m boundary. The rock mass deformability value used in the model is summarized in Table 10.

5.3 Stress and deformation analysis

The numerical modelling was done using a finite element method through rock science Phase2 software. The stress and deformation were predicted from the model. On the time of excavation and after the excavation of cavern, the stresses on the rock mass will be readjusted after distribution around the periphery of excavated surface. The amount of load carried by rock mass before excavation will be transferred to remaining mass once the excavation takes place. The stresses caused by the excavation depend on various factors, including the size and orientation of the opening, as well as the magnitude and direction of the principal stresses in the surrounding rock mass (Nilsen and Palmström 2000).

The data were measured using the multipoint extensometer after each excavation sequence and a comparison was made between the predicted model and the extensometer data. Analysis was done for each layer. At first, stress analysis was done at the powerhouse cavern before, during, and after excavation from the model. To date, only the first layer of the powerhouse cavern of Upper Trishuli-1 is excavated. So, the predicted displacement from the model after the first layer excavation from Ex1 is compared with the extensometer Ex1 and verified, and finally further analysis was made on different layers of the powerhouse cavern up to the final excavation. The support data used in the model was based on the project support data installed in the field. The displacement was predicted at five extensometers namely Ex1, Ex2, Ex3, Ex4, and Ex5 on each excavation layer.

The 2m depth from the excavated layer is taken as blasting damage zone where low deformation modulus was estimated using Hoek and Diederichs (2006) then other areas, so some stresses are distributed radially into the undisturbed or less disturbed rock mass.

From the numerical model, the stress distribution of major principal stress(σ_1) at Ex0 is constant before excavation and gradually increases with the excavation progress and at the final stage the major principal stress is lower at the surface of the excavation and increases sharply at 2-7m depth and again gradually decreases. The major principal stress is high at the crown whereas lower at the sidewalls (Figure 14) near the excavation boundary. Similarly, the distribution of minimum principal stress decreases along the excavation boundary and gradually increases with an increase in depth (Figure 16). Also, the mean stress before excavation is constant and increases with excavation stages (Figure 18). The mean stress is maximum at the crown and minimum at the walls along the excavation periphery. The stress concentration is mostly at the roof and bottom of the powerhouse cavern due to overstressing. As the disturbed zone has lower deformation modulus than rest of the area, the stresses are distributed at the transition between the disturbed zone and rest of rock mass.

The displacement yield at the powerhouse cavern roof at chainage 0+39m, on Ex1 after first layer excavation shows the relative displacement is similar up to 5m depth but the two points are not similar which is due to the presence of the weak band with quartz vein inclusion and sagging of the joint as shown in probe hole logging (Figure 9). The displacement increases with the excavation stages and at the final stage, the maximum displacement at Ex1 is 68 mm. Also, from the model the displacement increases up to 115 mm at the side wall (Ex4,

Ex5) after the final layer excavation. The total displacement from the extensometer and model follows a similar trend with a small change in coefficient and exponent. The relative displacement equation from extensometer data and from the model is given in Table 16 below.

The displacement yield at the powerhouse cavern roof at chainage 0+67m on Ex1 after first layer excavation (Table 13) also have relative displacement is similar up to 5m depth but the from 5 to 10 m shows the high deformation which is to the presence of the weak band with quartz vein inclusion and a clay band is present at 6.7m depth probe hole logging (Figure 9). The displacement increases with the excavation stages and at the final stage, the maximum displacement at Ex1 is 68 mm. Also, from the model the displacement increases up to 148 mm at the side wall (Ex4, Ex5) after the final layer excavation. The total deformation around the southern wall is higher than in other areas due to another cavern nearer to it.

Table 16 Relative displacement curve from the extensometer and numerical model

Total displacement versus distance		
Chainage(m)	Extensometer	Model
0+39	$y = 50.97e^{-0.183x}$	$y = 43.788e^{-0.08x}$
	$R^2 = 0.9954$	$R^2 = 0.9379$
0+67	$y = 53.348e^{-0.126x}$	$y = 49.625e^{-0.084x}$
	$R^2 = 0.9773$	$R^2 = 0.9867$

Sapigni et al. (2002) compare the predicted and measured deformation of the powerhouse cavern using FEM and DEM code and found that the predicted deformation is reasonably closer to the measured deformation. A similar case is found during the study for the depth up to 5m, however 10 and 17m deformation depths from the model and the extensometer does not match which may be due to the model constraints on variegated geology and its structure.

The study by Zhou et al. (2012) focused on investigating the deformation behaviour of large underground openings under different support conditions, using the Jinping II Hydropower Station in China as a case study. The results of the study showed that the deformation of the underground openings was significantly influenced by the support conditions, particularly the spacing and type of rock bolts used. The numerical simulations also showed that the use of a more robust support system resulted in reduced deformation and improved stability of

the underground openings. During the analysis of the deformation of the Powerhouse cavern of Upper Trishuli-1, the support system was adopted in the model after each excavation stages.

B.K. and Panthee (2019), analyse the stress and deformation of the powerhouse cavern of the Rasuwagadhi hydroelectric project on quartzite rock using 2D FEM and 3D BEM and found that the data obtained from 2D FEM, 3D BEM, and observed data shows the similar result. The study showed that the FEM analysis provided a reliable prediction of the deformation behaviour of the tunnel, and the results were consistent with the extensometer data. The study also showed that the deformation behaviour of the tunnel was significantly influenced by the geological conditions of the surrounding rock mass, as well as the excavation method and the support system used. The study verifies the deformation of the cavern is like extensometer data and the model executed by 2D and 3D FEM.

Table 17 Comparison of predicted and observed total displacement of Powerhouse cavern of Upper Trishuli-1 after first layer excavation from the model and multipoint extensometer (Ex1).

Chainage	0+39 m		0+67 m	
Depth of anchor(m)	Displacement(mm)			
	Model	Extensometer and Convergence(mm)	Model	Extensometer and Convergence(mm)
0	52	52.5	54.66	54.8
2	36.58	35.5	42.42	42.15
5	24.76	22.7	30.20	32.91
10	19.62	7.03	18.74	10.68
17	11.00	2.4	12.08	7.38

The deformation is maximum at an anchor depth of 5 to 10m section which is due to the sagging of joints and inclusion of hydrothermal veins at the weak band. A part of it, the effect of water on the weak band and small clay band on the crown is a possible reason for high deformation at 5 to 10m depth. After complete excavation, the displacement is predicted from the model at Ex2, Ex3, Ex4, and Ex5 at the side walls and the maximum displacement at Ex5 is shown in Table 17 below. The total displacement is maximum at the pillar between the powerhouse cavern and the transformer cavern.

Table 18 Total displacement after final layer excavation from the model at Ex5

Chainage(m)	0+39	0+67
Depth of anchor(m)	Displacement(mm)	
	Model	Model
0	115.84	155
2	109.87	139.13
5	99.74	123.42
10	86.96	110.7
17	63.58	87.38

Aghababaei et al. (2019). In this study, a large cavern was excavated in a hard rock formation and monitored using multipoint extensometers. A two-dimensional numerical model of the excavation was developed using the Phase2 software, and the results were compared with the field data. The numerical modelling showed that the maximum displacements occurred at the corners of the excavation, where the stress concentrations were the highest. The results also showed that the deformation behaviour of the rock mass was highly dependent on the geological structure and strength properties of the rock mass. The comparison of the numerical modelling results with the multipoint extensometer data showed good agreement, indicating the accuracy of the model in predicting the deformation behaviour of the rock mass. However, it is important to note that the accuracy of the numerical modelling results is highly dependent on the accuracy of the input data, including the geological and geomechanical parameters. Therefore, obtaining accurate data through geological investigations and laboratory testing is critical for developing reliable numerical models. On comparing with the deformation analysis on present study form numerical model, the deformation is maximum at the crown during the first layer of excavation and slowly increased while excavating further. Similarly, after final excavation the deformation of powerhouse cavern is centred towards the southern wall. The study agrees about accuracy of numerical modelling is based on the input parameter mostly, the selection of deformation modulus is very crucial due to the high sensitivity as explained by Panthee et al. (2016). Even a small change in Q or RMR value makes a bigger fluctuation in the deformation modulus and a change in deformation modulus will make a different in numerical analysis. So, care should be taken during the study and analysis of input parameters.

CHAPTER VI

CONCLUSION AND RECOMMENDATIONS

6.1 CONCLUSIONS

The deformation study was done at the powerhouse cavern of Upper Trishuli-1. The deformation at powerhouse occurs due to the excavation process, which involves removing a significant amount of rock mass from the ground. This removal of rock mass causes stress redistribution in the surrounding rock mass, which can lead to deformation. Additionally, the stress concentration is mostly at the roof and sidewalls. The instrument data of powerhouse cavern shows the deformation not in sequence basically at 5 to 10 m depth which have higher deformation than other areas.

Geologically, Upper Trishuli-1 lies at the Lesser Himalayan succession. The study area consists of a schist unit and a gneiss unit. The powerhouse cavern consists of rock of schist unit which is psammitic schist interbedded with pelitic schist. The 2m of the powerhouse cavern consists of disturbed rock mass. The roof of the powerhouse cavern has a sequence of strong psammitic schist with weak pelitic schist with the inclusion of a quartz vein where the quartz vein is irregular around the area. The presence of such weak and highly schistose and fractured rock mass in between forms deformation along the roof.

The rock mass classification was done at a 5m interval in the powerhouse cavern using Bieniawski's geomechanics classification system RMR (1989), Tunnel Quality Index Q Barton et al. (1974), and Geological strength index GSI Sonmez and Ulusay (1999). where Q-value ranges from 4.15 to 7.21 and RMR ranges from 39 to 56 and GSI ranges from 39 to 52. The rock mass classification shows the rock to be fair rock mass.

The uniaxial compressive strength (UCS) test was done in a laboratory using IS code and the maximum and minimum UCS is 81.03 MPa and 38.96 MPa with an average UCS of 60 MPa was calculated.

The deformation modulus of rock mass was estimated using the empirical relation given by Bieniawski (1989), Serafim and Pereira (1983), Grimpstand and Barton (1993), Hoek et al (2002), Hoek and Diederichs (2006) and ranges between 2.51 to 27.03 GPa. The deformation

modulus was used using the relation given by Hoek and Diederichs (2006) using the disturbance factor 0.5 for the 2m depth and 0.3 for remaining depth.

The instrumentation was installed at the central crown at two chainages which is Ch 0+39 m and Ch 0+67m. From the monitoring data at Ex1, the maximum and minimum deformation at crown is 49.58mm and 2.92 mm respectively and the convergence data shows the deformation at surface is 52.5 mm at Ch 0+39 m. Similarly, From the monitoring data at Ex1, the maximum and minimum deformation at crown is 47.42mm and 7.38 mm respectively and the convergence data shows the deformation at surface is 54.66mm at Ch 0+67 m. The deformation at each anchor depth does not shows the similar characteristics as the 5 to 10 m depth shows the maximum deformation.

Phase2 software was used for the analysis of stress and deformation of the powerhouse cavern. The maximum and minimum deformation predicted from the model at Ch 0+39 m is 52.9 mm and 11mm respectively. Also, the maximum and minimum deformation predicted from the model at Ch 0+67 m is 54.66 mm and 12.08mm respectively. Similarly, the deformation predicted from the model is maximum towards the walls of cavern after the complete excavation of tunnel which is measured by Ex5. The maximum is at the southern wall which is 139.13mm at the surface.

On comparing, the deformation predicted from model shows the gradual decrease of deformation with the depth but the deformation measured from extensometer does not shows similar behaviour. The relative displacement curve at Ch 0+39 m from extensometer data Ex1 follows the equation $y=50.97e^{-0.183x}$ where x is the depth of the extensometer with trendline reliability ($R^2=0.9915$). Similarly, the displacement from the model follows the equation $y=43.788e^{-0.081x}$ with trendline reliability ($R^2=0.9379$). The R-squared values indicate the goodness of fit of the regression models to the data. A high R-squared value indicates that the model fits the data well, while a low R-squared value indicates that the model does not fit the data well. Comparing these two equations, we can see that the first equation has a steeper slope (-0.183) compared to the second equation (-0.08), which means that the displacement decreases more rapidly with distance from the extensometer whereas the modelling possesses slow displacement while increasing the distance.

Similarly, at Ch 0+67 the relative displacement curve from extensometer data follows the equation $y=53.348e^{-0.126x}$ where x is the depth of the extensometer with trendline

reliability ($R^2=0.9773$). Similarly, the displacement from the model follows the equation $y=49.625e-0.084x$ with trendline reliability ($R^2=0.9773$). A high R-squared value indicates that the model fits the data well, comparing these two equations, we can see that the first equation has a steeper slope (-0.126) compared to the second equation (-0.084), which means that the displacement decreases more rapidly with distance from the extensometer whereas the modelling possesses slow displacement while increasing the distance. One possible reason for this difference could be that the model assumptions do not fully capture the complexity of the deformation behaviour of the cavern. It is possible that there are factors that are not taken into account by the model, such as non-linear behaviour of the rock mass or the presence of fractures or other geological structures that affect the deformation behaviour.

6.2 RECOMMENDATIONS

- The study of blasting damage zone and the observed deformation will be more reliable for curve characteristics.
- This study only considers the deformation measured in the first layer of excavation however the measurements should be compared after all layers of excavation.
- The displacement on an excavated area is influenced by time so, the time dependent stress and deformation analysis is recommended
- A 3D numerical modelling is necessary for the proper analysis of the powerhouse cavern.
- The deformation analysis was carried out using 2D FEM. For some specific joint sets, it can be done with the discrete element method (DEM) using UDEC.
- It is important for the user to have a good understanding of the assumptions made in each analysis method, in order to properly interpret the results. Knowing the limitations of the method is crucial for accurate interpretation. It is recommended to compare it to similar projects whenever possible.

REFERENCES

- Aghababaei, M., Sereshki, F., and Shahriar, K., 2019. Numerical modelling of deformation behaviour of an underground cavern and its correlation with multipoint extensometer measurements. *Journal of Rock Mechanics and Geotechnical Engineering*, v. 11, pp. 157-168.
- B. K, N., and Panthee, S., 2020. Stress and deformation analysis of powerhouse cavern of Rasuwagadhi hydroelectric project, Rasuwa, Central Nepal. *Lowland Technology International*, v. 22 (1), pp. 20 – 26.
- Barton, N., 2002. Some new Q value correlations to assist in site characterization and tunnel design. *International Journal of Rock Mechanics and Mining Sciences*, v. 39, pp. 185–216.
- Barton, N., Lien, R., and Lunde, J., 1974. Engineering Classification of Rock Masses for the Design of Tunnel Support. *Rock Mechanics*, v. 6, pp. 183-236.
- Beniawski, Z. T., 1978. Determining rock mass deformability: Experience from case histories. *International Journal of Rock Mechanics and Mining Sciences and Geomechanics*, v. 15, pp. 237–248.
- Bieniawski, Z. T., 1973. Engineering Classification of jointed Rock Masses. *Transaction of the South African Institution of Civil Engineers*, v. 15, pp. 335-344.
- Bieniawski, Z. T., 1974. Estimating the strength of rock materials. *Journal of the Southern African Institute of Mining and Metallurgy*, v. 74 (8), pp. 312–20
- Bieniawski, Z. T., 1989. *Engineering Rock Mass Classifications: A complete manual for engineers and geologists in mining, civil, and petroleum engineering*. John Wiley, Rotterdam, p. 251.
- Cai, M., Simons, R. M. B., and Wu, H., 2004. 2D FEM analysis of underground cavern stability in hard rock masses. *International Journal of Rock Mechanics and Mining Sciences*, v. 41 (3), pp. 466-467.

- Chen, Y. J., Zhang, X. Y., and Cui, Y. J., 2005. Stability analysis of large underground caverns in Jinping II hydropower station using FEM. *Chinese Journal of Rock Mechanics and Engineering*, v. 24, pp. 4326-4331.
- Colchen, M., Le Fort, P., and Pêcher, A., 1980. Carte géologique Annapurna- Manaslu-Ganesh Himal du Nepal Echelle, Centre National de la Recherche Scientifique, Paris, 136 p.
- Deere, D. U., Hendron, A. J., Patton, F. D., and Cording, E. J., 1967. Design of Surface and Near Surface Construction in Rock. *Proceedings of 8th US symposium Rock Mechanics*, AIME, New York, pp. 237–302.
- Gansser, A., 1964. *Geology of the Himalayas*. Interscience Publication, Wiley, New York, 289 p.
- Ghalan, K., 2022. Stability Assessment of the Underground Powerhouse for the Tamakoshi V Hydroelectric Project. Master's thesis, Norwegian University of Science and Technology, Department of Geoscience and Petroleum, 143 p.
- Grimstad, E., and Barton, N., 1993. Updating the Q-system for NMT. *Proceedings of international symposium on sprayed concrete- modern use of wet mix sprayed concrete for underground support*, Fagernes , Kompenm Opsahl and Berg (Eds.) Norway, Norwegian Concrete Association, Oslo, 20 p.
- Hoek, E. and Diederichs, M. S., 2006. Empirical estimation of rock mass modulus. *International Journal of Rock Mechanics and Mining Sciences*, v. 36, pp. 203–215.
- Hoek, E., and Brown, E. T., 1997. Practical estimates of rock mass strength. *International Journal of Rock Mechanics and Mining Sciences*, v. 34 (8), pp. 1165-1186.
- Hoek, E., and Brown, E. T., 2019. The Hoek–Brown failure criterion and GSI– 2018 edition. *Journal of Rock Mechanics and Geotechnical Engineering*, v. 11 (3), pp. 445–463.
- Jia, W., 2020. Deformation analysis of powerhouse cavern using 2D FEM and extensometers. *Tunnelling and Underground Space Technology*, v. 96, pp. 103-255.
- K. C, J., and Paudyal, K. R., 2019. Characteristics and field relation of Ulleri Augen Gneiss to country rocks in the Lesser Himalaya: A case study from Syaprubesi-Chhyamthali area, central Nepal, *Journal of Nepal Geological Society*, v. 58, pp. 89-96.

- Khandelwal, M., Singh, T. N., and Sastry, V. M., 2018. Estimation of rock strength parameters of schistose rock masses using empirical correlations: A case study from the Lesser Himalayas, India. *Geotechnical and Geological Engineering*, v. 36(3), pp. 1363-1373.
- Laubscher, D. H., 1977. Geomechanics classification of jointed rock masses – Mining Applications. *Transactions, Institute of Mining and Metallurgy*, 86 p.
- Laubscher, D. H., 1984. Design aspects and effectiveness of support systems in different mining conditions. *Transactions, Institute of Mining and Metallurgy*, 93p.
- Laubscher, D. H., and Page, C. H., 1990. The Design of Rock Support in High Stress or Weak Rock Environments. *Proceeding of 92nd Canadian institute of Mining and Metallurgy*, Ottawa, 91 p.
- Laubscher, D. H., and Taylor, H. W., 1976. The importance of geomechanics classification of jointed rock masses in mining operation. *Proceeding of Symposium of Exploration for Rock Engineering*, Johannesburg, v. 1, pp. 119-135.
- NEA, 2022. Electricity generation of power plants and projects. *NEA annual report*, 128 p.
- Nilsen, B., and Palmström, A., 2000. *Engineering Geology and Rock Engineering: Handbook*. Norwegian Group for Rock Mechanics.
- Palmstrom, A., 1982. The volumetric joint count - a useful and simple measure of the degree of rock jointing. *Proceedings of the 4th Congress of the International Association of Engineering Geology*, Delhi, v. 5, pp. 221-228.
- Panthee, S., Singh, P. K., Kainthola, A., Das, R., and Singh, T. N., 2016. Comparative study of the deformation modulus of rock mass. *Bulletin of Engineering Geology and the Environment*, v. 77 (2), pp. 751–760.
- Panthi, K. K., 2006. Analysis of engineering geological uncertainties related to tunnelling in Himalayan rock mass conditions. *Doctoral thesis*, Norwegian University of Science and Technology, Department of Geology and Mineral Resources Engineering, Trondheim, Norway. 155 p.

Rai, S. M., 2011. Lithostatigraphy of Nawakot Complex (Lesser Himalayan Sequence) from Malekhu area (south-west) to Syabrubensi area (north-east) along the Trishuli River, central Nepal Himalaya, Nepal Geological Society, v. 42, pp. 65-74.

Sapigni, M., La Barbara, G., and Ghirotti, M., 2002. Engineering Geological Characterization and Comparison of predicted and measured deformations of a cavern in the Italian Alps, Engineering Geology, v. 69, pp. 47-62.

Serafim, J. L. and Pereira, J. P., 1983. Considerations of the geomechanics classification of Bieniawski. Proceedings of International Symposium on Engineering Geology Underground Construction, Lisbon, pp. 1133-1142.

Shrestha, G. L., 2021. Rock Engineering Handbook on Design of Tunnel and Other Underground Structures. Global Nepal Publication, Nepal. 237 p.

Shrestha, H. M., 1998. Facts and figures about hydropower development in Nepal. Journal of Water, Energy and Environment, v. 20, pp. 1-5.

Sonmez, H., Ulusay, R., 1999. Modifications to the geological strength index (GSI) and their applicability to stability of slopes. International Journal of Rock Mechanics and Mining Sciences, v. 36, pp. 743–760.

Stöcklin, J., 1980. Geology of Nepal in its regional frame. Journal of Geological Society, London, v. 137, pp. 1–34

Stöcklin, J., and Bhattarai, K. D., 1977. Geology of Kathmandu area and Central Mahabharat range: Nepal Himalaya.

Terzaghi, K., 1946. Rock defects and loads on tunnel support: Introduction to Rock Tunnelling with Steel Supports. Eds: R. V. Proctor and T. L. White, Commercial Sheering and Stamping Co., Youngstown, Ohio, U.S.A., 271 p.

Varadarajan, A., Sharma, K. G., Desai, C. S., and Hashemi, M., 2007. Analysis of a powerhouse cavern in the Himalaya. International Journal of Rock Mechanics and Mining Sciences, v. 44 (2), pp. 197-214.

- Venkatachalam, G., and Singh, V. K., 2017. Effect of mineralogical composition on the deformation behaviour of schist under triaxial compression. *Journal of Rock Mechanics and Geotechnical Engineering*, v. 9(3), pp. 508-515.
- Wang, X., Jiang, Y., Liu, Y., and Zhang, Y., 2014. Analysis of deformation behaviour of a large underground cavern using a 2D FEM model. *Arabian Journal of Geosciences*, v. 7 (5), pp. 185-194.
- Winn, K., Melvin, N., and Wong, L. N. Y., 2017. Stability analysis of underground storage cavern excavation in Singapore. *Procedia Engineering, Elsevier*, v. 191, pp. 1040-1047.
- Wong, L. N., 2009. Numerical simulation of the deformation behaviour of large underground excavations using the finite element method. *Tunnelling and Underground Space Technology*, v. 24 (3), pp. 261-278.
- Zhang, W. H., Wang, Y. L., and Liu, B. H., 2008. Numerical simulation of deformation and stability of large underground cavern in Xiluodu hydropower station. *Water Resources and Power*, v. 26 (6), pp. 94-97
- Zhou, W., 2012. Deformation of large underground openings under different support conditions: A case study from the Jinping II Hydropower Station, China. *Engineering Geology*, v. 139, pp. 27-38.

APPENDICES

APPENDICES

Appendix A Rock Mass Rating System (After Bieniawski 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Stickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)									
Strike and dip orientations			Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable		
Ratings	Tunnels & mines	0	-2	-5	-10	-12			
	Foundations	0	-2	-7	-15	-25			
	Slopes	0	-5	-25	-50				
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS									
Rating	100 ← 81		80 ← 61	60 ← 41	40 ← 21	< 21			
Class number	I		II	III	IV	V			
Description	Very good rock		Good rock	Fair rock	Poor rock	Very poor rock			
D. MEANING OF ROCK CLASSES									
Class number	I		II	III	IV	V			
Average stand-up time	20 yrs for 15 m span		1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span			
Cohesion of rock mass (kPa)	> 400		300 - 400	200 - 300	100 - 200	< 100			
Friction angle of rock mass (deg)	> 45		35 - 45	25 - 35	15 - 25	< 15			
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions									
Discontinuity length (persistence)	< 1 m		1 - 3 m	3 - 10 m	10 - 20 m	> 20 m			
Rating	6		4	2	1	0			
Separation (aperture)	None		< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm			
Rating	6		5	4	1	0			
Roughness	Very rough		Rough	Slightly rough	Smooth	Stickensided			
Rating	6		5	3	1	0			
Infilling (gouge)	None		Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm			
Rating	6		4	2	2	0			
Weathering	Unweathered		Slightly weathered	Moderately weathered	Highly weathered	Decomposed			
Ratings	6		5	3	1	0			
F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**									
Strike perpendicular to tunnel axis				Strike parallel to tunnel axis					
Drive with dip - Dip 45 - 90°		Drive with dip - Dip 20 - 45°		Dip 45 - 90°		Dip 20 - 45°			
Very favourable		Favourable		Very unfavourable		Fair			
Drive against dip - Dip 45-90°		Drive against dip - Dip 20-45°		Dip 0-20 - Irrespective of strike*					
Fair		Unfavourable		Fair					

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

** Modified after Wickham et al (1972).

Appendix B Tunnelling Quality Index Q (After Barton et al 1974)

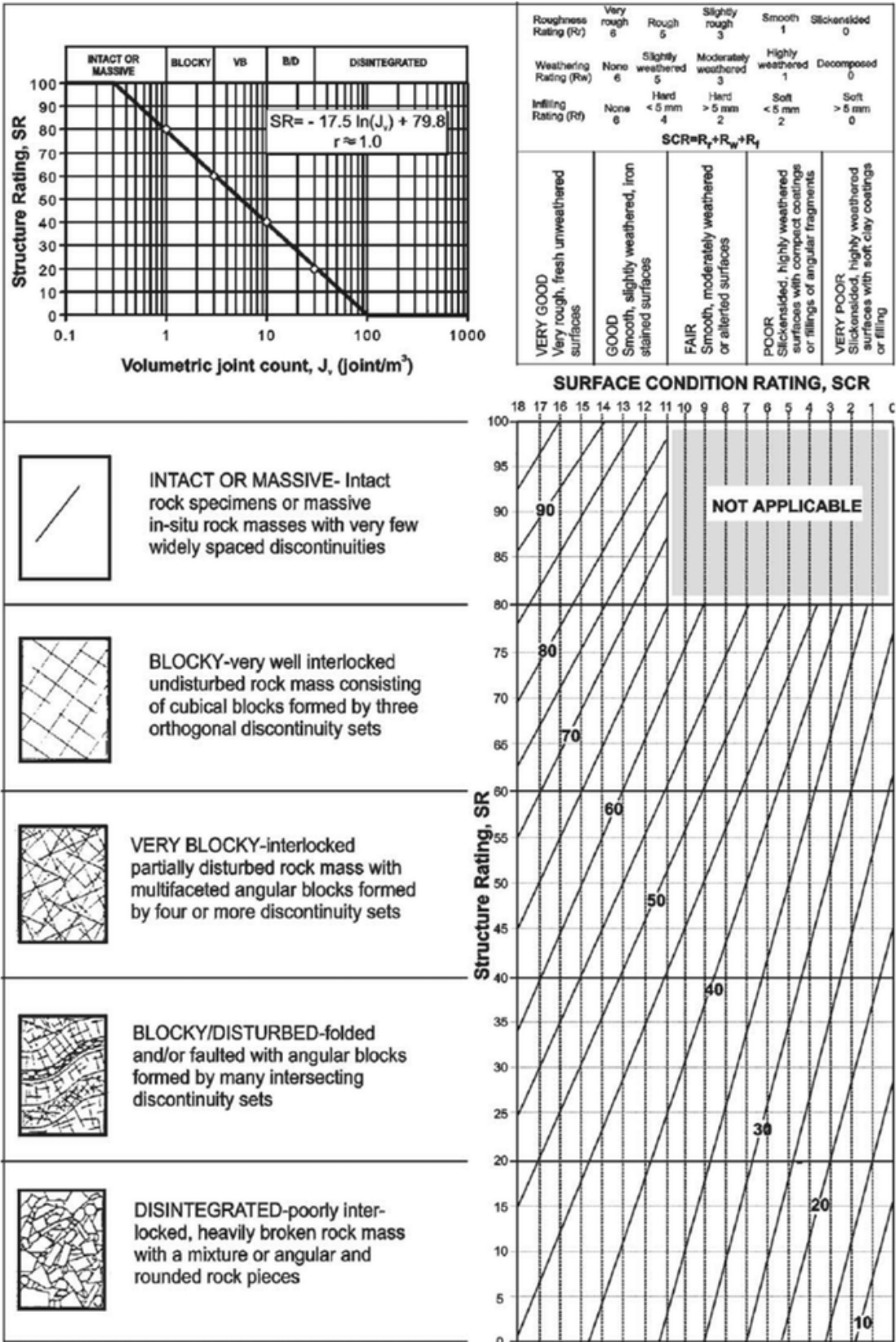
1. Rock Quality Designation		RQD	Grade
A	Very poor	0 - 25	
B	Poor	25 - 50	
C	Fair	50 - 75	
D	Good	75 - 90	
E	Excellent	90 - 100	
Note: i) Where RQD is reported or measured as ≤ 10 (incl. 0), a nominal value of 10 is used to evaluate Q. ii) RQD intervals of 5, i.e., 10, 100, 95, 90, etc., are sufficiently accurate.			
2. Joint set number		J_n	
A	Massive, none of few joints	0.5 - 1.0	
B	One joint set	2	
C	One joint set plus random joints	3	
D	Two joint sets	4	
E	Two joint sets plus random joints	6	
F	Three joint sets	9	
G	Three joint sets plus random joints	12	
H	Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	
J	Crushed rock, earthlike	20	
Note: For intersections, use $(0.5 \times J_n)$. For partings, use $(2.0 \times J_n)$.			
3. Joint roughness number		J_r	
a) Rock-wall contact, and			
b) Rock-wall contact before 10 cm shear			
A	Discontinuous joints	4	
B	Rough or irregular, undulating	3	
C	Smooth undulating	2	
D	Slack-sided, undulating	1.5	
E	Rough or irregular, planar	1.5	
F	Smooth, planar	1.0	
G	Slack-sided, planar	0.5	
Note: Description refers to small scale features, and intermediate scale features, in that order.			
c) No rock-wall contact when sheared			
H	Zone containing clay minerals thick enough to prevent rock wall contact	1.0	
I	Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1.0	
Note: Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. $J_r = 0.5$ can be used for planar slack-sided joints having lineations, provided the lineations are oriented for minimum strength.			
4. Joint alteration number		J_a approx	J_a
a) Rock wall contact (no mineral fillings, only coatings)			
A	Tightly healed hard non-softening, impermeable filling, i.e., quartz or epidote	-	0.75
B	Unhealed joint walls, surface staining only	25 - 35°	1.0
C	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25 - 30°	2.0
D	Silty or sandy clay coatings, small clay fraction (non-softening)	20 - 25°	3.0
E	Softening or low-friction clay mineral coatings, i.e., kaolinite or mica. Also chlorite, talc, gypsum, graphite, etc., and small quantities of swelling clays	8 - 16°	4.0
b) Rock wall contact before 10 cm shear (thin mineral fillings)			
F	Sandy particles, clay-free disintegrated rock, etc.	25 - 30°	4.0
G	Strongly over-consolidated, non-softening clay mineral fillings (continuous, but < 5mm thick)	16 - 24°	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, but < 5mm thick)	12 - 16°	8.0
J	Swelling clay fillings, i.e. montmorillonite (continuous, but < 5mm thick)	6 - 12°	8 - 12
Note: Value of J_a depends on percent of clay-size particles, and access to water, etc.			
c) No rock wall contact when sheared (thick mineral fillings)			
K	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6 - 24°	6, 8, or 8 - 12
N	Zones or bands of silty or sandy clay, small clay fraction (non-softening)	-	5
O	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6 - 24°	10, 13, or 13 - 20
P			
Q			

5. Joint water reduction factor		Water pressure (MPa)	Q_w	Grade
A	Dry excavations or minor inflow, i.e., < 5 l/min locally	< 0.1	1	
B	Medium inflow or pressure, occasional outwash of joint fillings	0.1 - 0.25	0.66	
C	Large inflow or high pressure in competent rock with unfilled joints	0.25 - 1.0	0.5	
D	Large inflow or high pressure, considerable outwash of joint fillings	0.25 - 1.0	0.33	
E	Exceptional high inflow or water pressure at blasting, decaying with time	> 1.0	0.2 - 0.1	
F	Exceptional high inflow or water pressure continuing without noticeable decay	> 1.0	0.1 - 0.05	
Note: Factors C to F are crude estimates. Increase Q_w if drains are installed.				
6. Stress reduction factor		SRF		
a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing clay or disintegrated rock, very loose surrounding rock (at any depth)	10.0		
B	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation ≤ 50 m)	5.0		
C	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)	2.5		
D	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)	7.5		
E	Single shear zones in competent rock (clay-free), (depth of excavation ≤ 50 m)	5.0		
F	Single shear zones in competent rock (clay-free), (depth of excavation ≥ 50 m)	2.5		
G	Loose open joints, heavily jointed or 'sugar cube', etc. (any depth)	5.0		
Note: Reduce these values of SRF by 25 - 50% if the relevant shear zones only influence but do not intersect the excavation.				
b) Competent rock, stress problems		σ_c / σ_1	σ_2 / σ_3	SRF
H	Low stress, near surface, open joints	> 200	< 0.01	2.5
J	Medium stress, favorable stress conditions	200 - 10	0.01 - 0.3	1
K	High stress, very tight structure. Usually favorable to stability, may be unfavorable for wall stability	10 - 5	0.3 - 0.4	0.5 - 2
L	Moderate slabbing after > 1 hour in massive rock	5 - 3	0.5 - 0.65	5 - 50
M	Slabbing and rock burst after a few minutes in massive rock	3 - 2	0.65 - 1.0	50 - 200
N	Heavy rock burst (strain burst) and immediate dynamic deformations in massive rock	< 2	> 1	200 - 400
Note: For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1 / \sigma_3 \leq 10$ reduce σ_3 to 0.75 σ_3 . When $\sigma_1 / \sigma_3 > 10$, reduce σ_3 to 0.5 σ_3 , where σ_1 = unconfined compressive strength, σ_2 and σ_3 are the major and the minor principal stress, respectively, and σ_3 is the maximum tangential stress estimated from elasticity theory. Few cases records are available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5.0 in such cases (see H).				
c) Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure		σ_2 / σ_3	SRF	
O	Mild squeezing rock pressure	1.5	5 - 10	
P	Heavy squeezing rock pressure	> 5	10 - 20	
Note: Cases of squeezing rock may occur for depth $H > 350 Q^{0.25}$. Rock mass compressive strength can be estimated from $q \approx 0.7 \gamma Q^{0.25}$ (MPa) where γ = rock density in kN/m ³ .				
d) Swelling rock, chemical swelling activity depending on pressure of water				
R	Mild swelling rock pressure	5 - 10		
S	Heavy swelling rock pressure	10 - 20		
Note: J and J_a classification is applied to the joint set or discontinuity that is least favorable for stability both from point of view of orientation and shear resistance τ (where $\tau = \sigma_n \tan(\phi)$).				
Determined Q-value:				
Rock Class:				

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Appendix C Geological Strength Index for Jointed rock mass from Sonmez and Ulusay (1999)



Appendix D Material constant based on the lithology classification (m_i) (Marinos and Hoek, 2001)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3)	Sandstones 17 ± 4	Siltstones 7 ± 2	Claystones 4 ± 2
			Breccias (19 ± 5)		Greywackes (18 ± 3)	Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

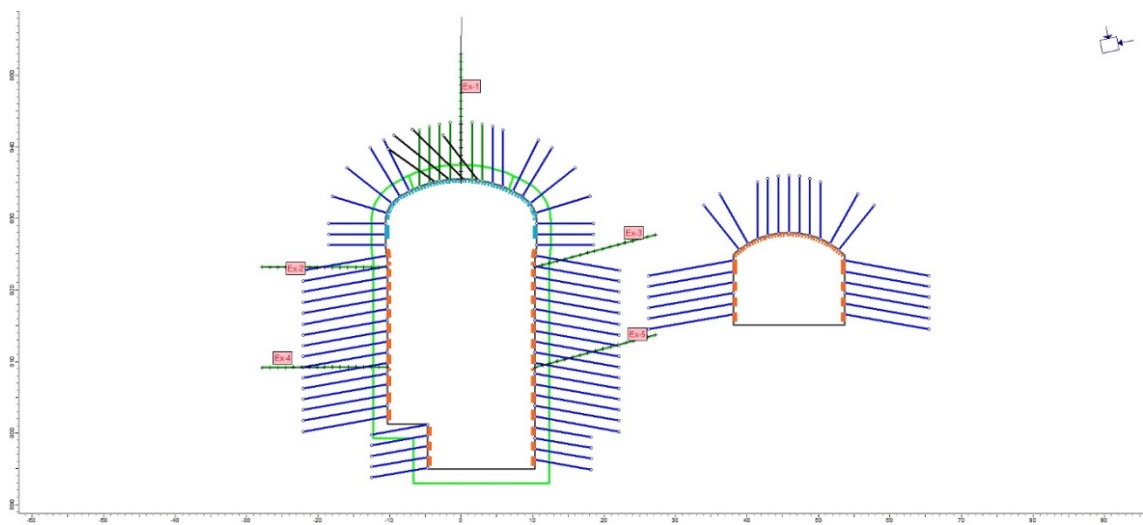
* Conglomerates and breccias may present a wide range of m_i values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine grained sediments.

**These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

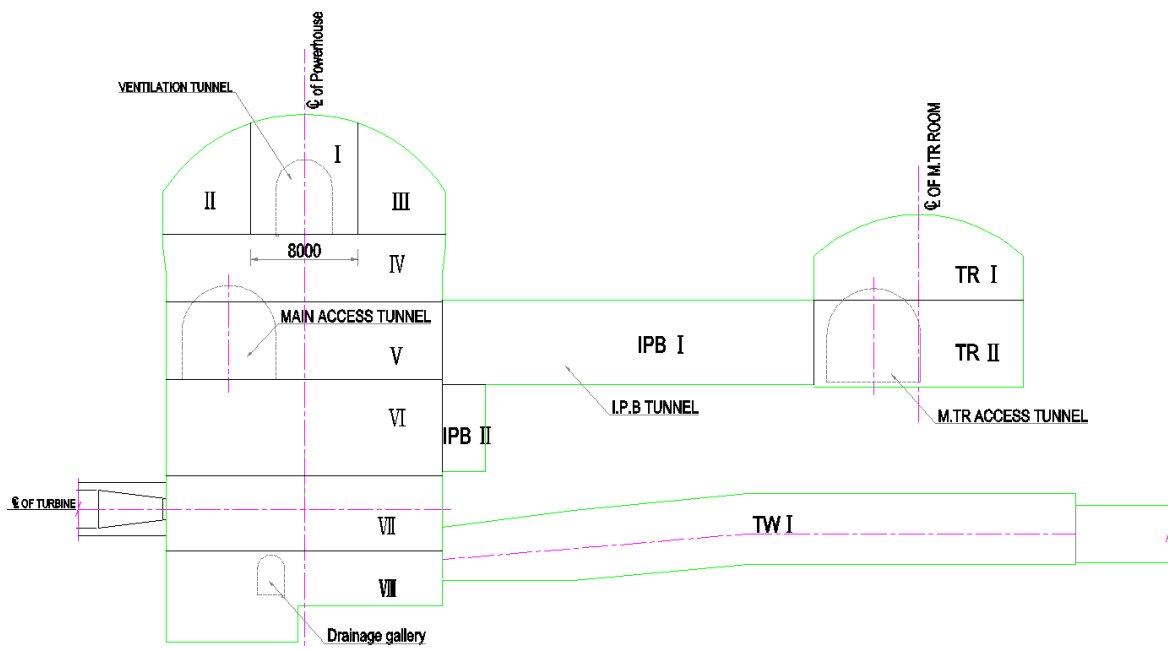
Appendix E Disturbance Factor (D). (E. Hoek, C. Carranza-Torres and B. Corkum 2002)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality-controlled blasting or excavation by a road-header or tunnel boring machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Case 1: Mechanical or manual excavation in poor quality rock masses gives minimal disturbance to the surrounding rock mass. Case 2: Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	Case 1: D = 0 Case 2: D = 0.5 with no invert
	Poor control of drilling alignment, charge design and detonation sequencing results in very poor blasting in a hard rock tunnel with severe damage, extending 2 or 3 m in the surrounding rock mass.	D = 1.0 at surface with a linear decrease to D = 0 at ± 2 m into the surrounding rock mass
	Small-scale blasting in civil engineering slopes results in modest rock mass damage when controlled blasting is used, as shown on the left-hand side of this photograph. Uncontrolled production blasting can result in significant damage to the rock face.	D = 0.5 for controlled presplit or smooth wall blasting with D = 1.0 for production blasting
	Case 1: Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. Case 2: In some weak rock masses, excavation can be carried out by ripping and dozing. Damage to the slopes is due primarily to stress relief.	Case 1: D = 1.0 for production blasting Case 2: D = 0.7 for mechanical excavation

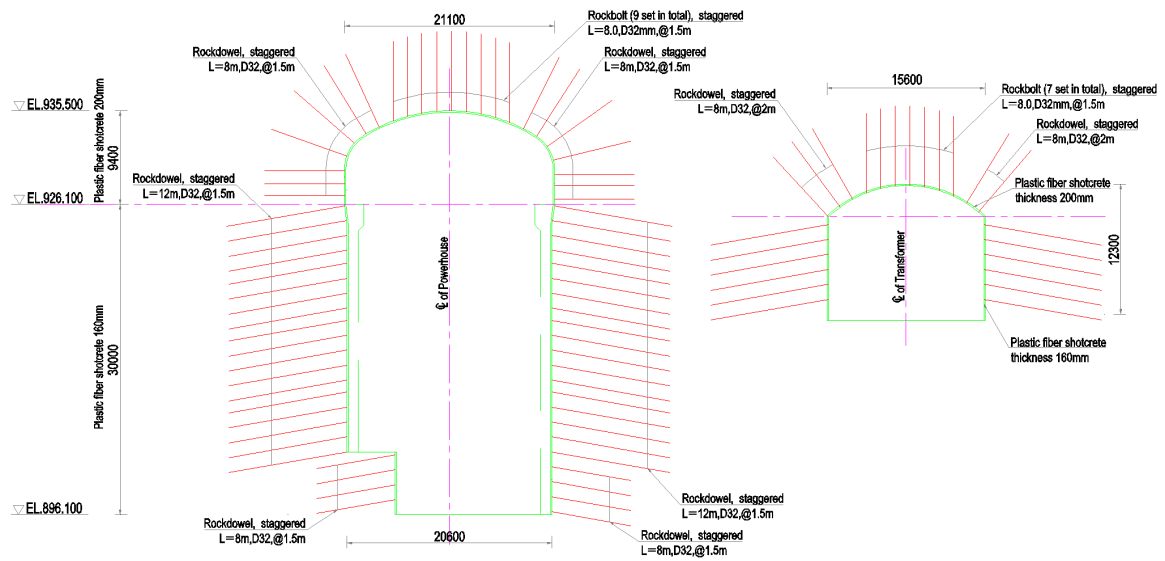
Appendix F Monitoring Extensometer location at Powerhouse cavern



Appendix G Excavation Plan of powerhouse cavern



Appendix H Support system of Powerhouse cavern



Appendix I Proposed supports of powerhouse and transformer cavern

Item	Unit	proposed supports	
		II	III
Shotcrete thickness	mm	150 mm	Roof :200 mm Side wall: 160 mm
Shotcrete strength	MPa	25	25
Powerhouse cavern supports		Roof and side walls: L=7.0m, ϕ32 mm/L=5.0m, ϕ28 mm staggered. Amongst, rock bolts are proposed in the central portion of the cavern crown.	Roof: L=8.0m, ϕ32 mm Side walls: L=12.0m, ϕ32 mm Lower side walls: L=8.0m, ϕ32 mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.

Item	Unit	proposed supports	
		II	III
Transformer cavern supports		Roof and side walls: L=7.0m, ϕ 28 mm/ L=5.0m, ϕ 25 mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.	Roof: L=8.0m, ϕ 32 mm Side walls: L=12.0m, ϕ 32 mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.
Rock dowels and/or rock bolts spacing within each row	m	2.0	1.5
Rock dowels and/or rock bolts row spacing	m	2.0	1.5

Appendix J Properties of Rock bolt (32 mm dia.) used in the model

Sn	Properties of Bolt (Rock)	Values
1	Tensile Capacity (MN)	0.3
2	Tributary Area (mm ²)	800
3	Bolt Modulus, E (MPa)	210000
4	Bond Strength (MN/m)	0.17
5	Out-of-plane spacing (m)	1.5
6	Bond Shear Stiffness (MN/m/m)	100
7	Residual Tensile Capacity (MN)	0.3

Compressive strength f_c' (Mpa)	Tensile Strength (Mpa)	Elastic modulus E_c (Mpa)	Unit Weight (kN/m ³)	Poisson ratio
25	2.5	23500	24.0	0.2

Appendix K Deformation Data from multipoint extensometer at Ch 0+39 m

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
31-Oct-22	0.00	0.00	0.00	0.00				
2-Nov-22	0.04	0.01	0.19	0.08				
3-Nov-22	2.15	1.93	0.66	2.99	0.00	0.00	0.00	0.00
4-Nov-22	4.70	3.48	0.73	3.41	0.15	0.13	0.27	0.27
5-Nov-22	6.68	3.86	0.97	6.16	0.29	3.36	3.65	3.65
6-Nov-22	6.54	4.11	1.11	7.42	0.33	3.81	4.14	4.14
7-Nov-22	7.33	5.01	1.22	8.42	0.48	6.64	7.12	7.12
8-Nov-22	7.25	5.15	1.30	8.88	0.52	8.02	8.53	8.53
9-Nov-22	7.12	7.70	1.31	9.09	-0.72	10.37	9.64	9.64
9-Nov-22	7.12	7.70	1.27	9.30	0.54	9.64	10.18	10.18
10-Nov-22	7.14	7.71	1.30	9.38	0.49	9.92	10.41	10.41
10-Nov-22	7.03	7.81	1.31	9.44	0.66	9.90	10.56	10.56
11-Nov-22	7.06	7.79	1.21	9.50	-0.63	11.31	10.69	10.69
11-Nov-22	7.05	7.79	1.18	9.68	0.70	10.05	10.75	10.75
11-Nov-22	7.04	7.81	1.24	9.63	-0.59	11.31	10.72	10.72
11-Nov-22	7.01	7.83	1.26	9.70	0.54	10.32	10.86	10.86
12-Nov-22	6.95	7.90	1.23	9.73	0.71	10.16	10.87	10.87
12-Nov-22	6.89	7.96	1.30	9.78	0.76	10.21	10.96	10.96
12-Nov-22	6.84	8.04	1.32	9.83	0.66	10.29	10.95	10.95
12-Nov-22	7.43	8.05	1.15	10.13	0.82	10.26	11.08	11.08
13-Nov-22	8.16	8.05	1.35	9.91	0.79	10.36	11.14	11.14
13-Nov-22	8.18	8.01	1.29	9.93	0.64	10.64	11.27	11.27
13-Nov-22	8.15	8.05	1.37	10.03	0.87	10.39	11.26	11.26
14-Nov-22	8.15	8.07	1.29	9.93	0.83	10.39	11.22	11.22
15-Nov-22	8.16	8.06	1.36	10.08	0.66	10.73	11.40	11.40
15-Nov-22	8.16	8.07	1.38	10.09	0.44	10.78	11.22	11.22
15-Nov-22	8.16	8.06	1.37	10.11	0.69	10.76	11.45	11.45
15-Nov-22	8.16	8.06	1.30	10.18	0.83	10.65	11.48	11.48
16-Nov-22	8.15	8.06	1.29	10.21	0.87	10.62	11.48	11.48
16-Nov-22	8.16	8.06	1.29	10.21	0.86	10.62	11.48	11.48
16-Nov-22	8.16	8.06	1.29	10.21	0.84	10.66	11.50	11.50
16-Nov-22	8.16	8.06	1.38	10.20	0.78	10.71	11.50	11.50
17-Nov-22	8.16	8.06	1.36	10.23	0.79	10.71	11.50	11.50
17-Nov-22	8.17	8.06	1.37	10.23	0.91	10.66	11.58	11.58
17-Nov-22	8.17	8.06	1.37	10.24	0.85	10.73	11.59	11.59

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
17-Nov-22	8.19	8.06	1.40	10.25	0.84	10.76	11.60	11.60
18-Nov-22	8.00	8.21	1.40	10.30	0.82	10.79	11.61	11.61
18-Nov-22	7.99	8.23	1.33	10.27	0.80	10.86	11.66	11.66
18-Nov-22	8.02	8.22	1.33	10.29	0.84	10.86	11.70	11.70
18-Nov-22	8.02	8.22	1.33	10.29	0.79	10.80	11.60	11.60
19-Nov-22	8.02	8.23	1.37	10.29	0.77	10.84	11.61	11.61
19-Nov-22	7.63	8.26	1.45	10.27	0.79	10.83	11.62	11.62
19-Nov-22	7.62	8.26	1.44	10.35	0.82	10.84	11.66	11.66
20-Nov-22	8.31	8.97	1.38	10.42	0.85	10.87	11.72	11.72
20-Nov-22	8.25	9.12	1.52	10.41	0.89	10.90	11.79	11.79
20-Nov-22	8.33	9.48	1.48	10.45	0.82	10.97	11.80	11.80
21-Nov-22	8.39	10.00	1.50	10.46	0.94	10.99	11.93	11.93
21-Nov-22	8.39	10.15	1.45	10.54	0.90	11.03	11.92	11.92
21-Nov-22	8.46	10.19	1.38	10.67	0.91	11.06	11.97	11.97
22-Nov-22	8.47	10.18	1.41	10.67	1.07	10.92	11.99	11.99
22-Nov-22	8.51	10.18	1.35	10.70	0.91	11.14	12.05	12.05
23-Nov-22	8.51	10.18	1.38	10.67	0.93	11.15	12.08	12.08
23-Nov-22	8.53	10.18	1.39	10.71	0.90	11.15	12.05	12.05
23-Nov-22	8.53	10.19	1.42	10.71	0.91	11.14	12.05	12.05
24-Nov-22	8.55	10.19	1.41	10.72	0.89	11.22	12.11	12.11
24-Nov-22	8.52	10.20	1.44	10.72	0.87	11.26	12.13	12.13
24-Nov-22	8.54	10.20	1.41	10.75	0.90	11.24	12.14	12.14
25-Nov-22	8.56	10.20	1.46	10.72	0.93	11.23	12.16	12.16
25-Nov-22	8.60	10.17	1.45	10.76	0.86	11.30	12.16	12.16
25-Nov-22	8.58	10.20	1.45	10.78	0.95	11.23	12.18	12.18
26-Nov-22	8.95	10.14	1.47	10.78	0.91	11.30	12.21	12.21
26-Nov-22	8.38	10.43	1.46	10.80	0.91	11.32	12.24	12.24
26-Nov-22	8.39	10.43	1.52	10.82	0.92	11.32	12.24	12.24
27-Nov-22	8.40	10.44	1.50	10.84	0.91	11.35	12.26	12.26
27-Nov-22	8.41	10.44	1.47	10.90	0.63	11.70	12.34	12.34
27-Nov-22	8.31	10.48	1.46	10.89	0.76	11.58	12.34	12.34
28-Nov-22	8.42	10.44	1.49	10.90	0.54	11.83	12.37	12.37
28-Nov-22	8.42	10.44	1.49	10.93	0.48	11.87	12.35	12.35
28-Nov-22	8.43	10.44	1.43	11.90	0.54	11.84	12.39	12.39
29-Nov-22	8.43	10.44	1.49	12.26	0.57	11.85	12.42	12.42
29-Nov-22	8.45	10.45	1.50	12.39	0.49	12.84	13.33	13.33

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
30-Nov-22	8.44	10.45	1.50	12.43	0.53	13.22	13.75	13.75
30-Nov-22	8.45	10.46	1.56	12.57	0.50	13.38	13.88	13.88
1-Dec-22	8.46	10.46	1.57	12.66	0.52	13.40	13.92	13.92
1-Dec-22	8.45	10.45	1.56	12.68	0.58	13.55	14.13	14.13
2-Dec-22	8.47	10.46	1.59	12.74	0.60	13.63	14.22	14.22
2-Dec-22	8.46	10.46	1.59	12.80	0.60	13.64	14.24	14.24
2-Dec-22	8.45	10.46	1.59	12.82	0.62	13.71	14.33	14.33
3-Dec-22	8.47	10.47	1.63	13.27	0.62	13.77	14.39	14.39
3-Dec-22	8.47	10.46	1.62	13.51	0.61	13.80	14.41	14.41
3-Dec-22	8.47	10.46	1.65	13.73	0.69	14.21	14.89	14.89
4-Dec-22	8.46	10.46	1.61	13.78	0.67	14.46	15.13	15.13
4-Dec-22	8.48	10.46	1.64	13.95	0.70	14.69	15.38	15.38
5-Dec-22	8.49	10.47	1.69	13.94	0.67	14.71	15.38	15.38
6-Dec-22	8.56	10.47	1.64	14.03	0.67	14.92	15.59	15.59
6-Dec-22	8.50	10.46	1.66	14.13	0.66	14.97	15.63	15.63
7-Dec-22	8.49	10.47	1.75	14.13	0.67	14.99	15.67	15.67
7-Dec-22	8.51	10.47	1.67	14.69	0.67	15.12	15.79	15.79
8-Dec-22	8.50	10.46	1.59	14.94	0.66	15.21	15.87	15.87
8-Dec-22	8.51	10.47	1.67	14.99	-0.75	17.11	16.36	16.36
9-Dec-22	8.51	10.47	1.62	15.22	0.72	15.81	16.53	16.53
9-Dec-22	8.51	10.47	1.67	15.22	0.71	15.95	16.67	16.67
10-Dec-22	8.51	10.47	1.66	15.30	0.74	16.10	16.85	16.85
10-Dec-22	8.52	10.47	1.78	16.54	0.69	16.21	16.89	16.89
11-Dec-22	8.51	10.47	1.73	16.76	0.71	16.25	16.96	16.96
11-Dec-22	8.51	10.47	1.73	16.91	0.88	17.44	18.32	18.32
11-Dec-22	8.54	10.48	1.72	17.71	0.85	17.64	18.49	18.49
12-Dec-22	8.52	10.48	1.64	17.96	0.87	17.77	18.64	18.64
13-Dec-22	8.53	10.48	1.28	18.71	0.85	18.58	19.43	19.43
13-Dec-22	8.53	10.48	1.16	19.02	0.74	18.85	19.60	19.60
14-Dec-22	8.52	10.49	1.38	19.93	0.33	19.67	19.99	19.99
15-Dec-22	8.53	10.49	1.12	20.39	0.28	19.90	20.18	20.18
17-Dec-22	8.54	10.48	1.12	20.40	-0.21	21.52	21.31	21.31
17-Dec-22	8.54	10.47	2.28	21.01	-0.46	21.98	21.52	21.52
17-Dec-22	8.56	10.48	2.28	21.05	-0.48	22.00	21.52	21.52
18-Dec-22	8.55	10.48	5.43	21.69	0.04	23.26	23.29	23.29
19-Dec-22	8.54	10.48	5.26	21.90	-0.35	23.69	23.33	23.33

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
19-Dec-22	8.55	10.48	5.28	21.92	0.99	26.13	27.12	27.12
19-Dec-22	8.56	10.48	9.97	22.35	1.00	26.17	27.16	27.16
19-Dec-22	16.92	10.47	10.85	22.46	0.31	26.89	27.19	27.19
20-Dec-22	16.69	10.69	13.10	23.05	3.16	29.17	32.32	32.32
20-Dec-22	16.69	10.69	13.68	23.13	3.60	29.71	33.32	33.32
21-Dec-22	16.68	10.70	15.72	23.58	4.17	31.98	36.15	36.15
21-Dec-22	16.72	10.69	16.14	23.66	4.39	32.43	36.81	36.81
22-Dec-22	16.68	10.69	16.84	24.02	5.06	34.25	39.31	39.31
22-Dec-22	16.67	10.70	17.44	24.24	5.19	34.60	39.80	39.80
23-Dec-22	16.67	10.70	17.46	24.21	5.43	35.43	40.86	40.86
23-Dec-22	16.67	10.71	17.53	24.23	5.56	36.11	41.68	41.68
23-Dec-22	16.66	10.71	17.61	24.25	5.56	36.11	41.67	41.67
24-Dec-22	16.55	10.82	17.64	24.26	5.67	36.10	41.76	41.76
24-Dec-22	16.54	10.83	17.65	24.28	5.75	36.11	41.86	41.86
24-Dec-22	16.55	10.83	17.73	24.30	5.78	36.12	41.90	41.90
24-Dec-22	16.53	10.83	18.00	24.42	5.63	36.31	41.93	41.93
24-Dec-22	16.51	10.84	18.08	24.46	5.71	36.32	42.03	42.03
25-Dec-22	16.54	10.84	18.20	24.49	5.78	36.64	42.42	42.42
25-Dec-22	16.54	10.84	18.16	24.52	5.88	36.66	42.54	42.54
25-Dec-22	16.38	10.95	18.39	24.60	5.86	36.83	42.69	42.69
25-Dec-22	16.40	10.98	18.58	24.69	5.82	36.87	42.68	42.68
26-Dec-22	16.40	10.96	18.58	24.69	5.94	37.05	42.99	42.99
26-Dec-22	16.40	10.98	18.72	24.76	5.92	37.35	43.27	43.27
27-Dec-22	16.40	10.98	18.73	24.90	5.92	37.35	43.27	43.27
27-Dec-22	16.40	10.98	18.79	24.85	6.03	37.44	43.48	43.48
27-Dec-22	16.40	10.98	18.85	25.00	6.03	37.61	43.63	43.63
28-Dec-22	16.40	10.98	18.89	25.01	6.03	37.61	43.64	43.64
28-Dec-22	16.41	10.97	18.95	24.96	6.21	37.63	43.85	43.85
28-Dec-22	16.40	10.97	18.93	25.08	6.14	37.77	43.91	43.91
29-Dec-22	16.40	10.98	19.41	25.33	6.06	37.86	43.92	43.92
29-Dec-22	21.69	10.90	19.45	25.37	6.23	37.78	44.02	44.02
30-Dec-22	21.69	10.90	19.74	25.52	6.36	38.38	44.74	44.74
30-Dec-22	21.71	10.88	19.79	25.63	6.31	38.52	44.82	44.82
31-Dec-22	21.73	10.88	20.04	25.81	6.38	38.88	45.26	45.26
31-Dec-22	21.73	10.89	20.03	25.82	6.41	39.01	45.42	45.42
31-Dec-22	26.87	10.57	20.17	25.83	6.47	39.38	45.85	45.85

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
1-Jan-23	27.01	10.42	20.14	25.90	6.46	39.38	45.85	45.85
1-Jan-23	27.01	10.41	20.39	25.90	6.49	39.51	46.00	46.00
1-Jan-23	27.24	10.19	20.28	26.03	6.55	39.49	46.04	46.04
1-Jan-23	33.11	10.18	20.28	26.07	6.62	39.67	46.29	46.29
2-Jan-23	33.20	10.02	20.30	26.15	6.64	39.68	46.31	46.31
2-Jan-23	33.20	10.01	20.30	26.17	6.52	39.83	46.36	46.36
2-Jan-23	33.18	10.01	20.31	26.17	6.59	39.86	46.45	46.45
3-Jan-23	33.30	9.88	20.33	26.22	6.61	39.85	46.46	46.46
3-Jan-23	33.29	9.89	20.36	26.22	6.47	40.01	46.48	46.48
3-Jan-23	33.30	9.87	20.38	26.26	6.54	40.01	46.55	46.55
3-Jan-23	33.29	9.87	20.43	26.27	6.56	40.01	46.57	46.57
4-Jan-23	33.30	9.85	20.57	26.28	6.63	40.00	46.63	46.63
4-Jan-23	33.30	9.85	20.55	26.30	6.55	40.15	46.70	46.70
4-Jan-23	33.31	9.85	20.55	26.33	6.68	40.18	46.85	46.85
4-Jan-23	33.25	9.85	20.54	26.32	6.67	40.18	46.85	46.85
5-Jan-23	33.29	9.85	20.50	26.42	6.70	40.17	46.88	46.88
5-Jan-23	33.27	9.86	20.54	26.43	6.68	40.18	46.86	46.86
6-Jan-23	33.31	9.86	20.66	26.43	6.53	40.38	46.92	46.92
6-Jan-23	33.29	9.85	20.65	26.45	6.58	40.39	46.97	46.97
6-Jan-23	33.28	9.86	20.64	26.48	6.70	40.39	47.09	47.09
7-Jan-23	33.30	9.85	20.68	26.48	6.71	40.39	47.10	47.10
7-Jan-23	33.28	9.89	20.64	26.49	6.71	40.40	47.11	47.11
7-Jan-23	33.30	9.86	20.71	26.48	6.71	40.45	47.16	47.16
7-Jan-23	33.27	9.86	20.78	26.52	6.73	40.40	47.13	47.13
8-Jan-23	33.28	9.85	20.80	26.53	6.78	40.41	47.19	47.19
8-Jan-23	33.26	9.85	20.84	26.55	6.73	40.58	47.30	47.30
8-Jan-23	33.52	9.61	20.72	26.53	6.76	40.58	47.34	47.34
8-Jan-23	33.27	9.84	20.70	26.61	6.81	40.58	47.39	47.39
8-Jan-23	33.26	9.85	20.71	26.61	6.66	40.59	47.25	47.25
8-Jan-23	33.27	9.85	20.69	26.61	6.71	40.59	47.31	47.31
9-Jan-23	33.27	9.84	20.70	26.61	6.72	40.59	47.31	47.31
9-Jan-23	33.26	9.85	20.89	26.67	6.68	40.62	47.30	47.30
10-Jan-23	33.26	9.85	20.92	26.70	6.71	40.60	47.31	47.31
10-Jan-23	35.91	9.85	20.92	26.70	6.82	40.74	47.57	47.57
10-Jan-23	36.04	9.69	20.89	26.68	6.87	40.76	47.62	47.62
10-Jan-23	36.04	9.69	20.88	26.68	6.87	40.76	47.62	47.62

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
10-Jan-23	36.04	9.68	20.90	26.75	6.83	40.75	47.57	47.57
11-Jan-23	36.03	9.69	20.86	26.81	6.82	40.75	47.57	47.57
12-Jan-23	36.02	9.68	20.96	26.80	6.84	40.81	47.65	47.65
12-Jan-23	36.02	9.68	20.93	26.82	6.85	40.81	47.66	47.66
12-Jan-23	36.02	9.68	20.96	26.86	6.84	40.92	47.76	47.76
13-Jan-23	36.02	9.65	20.96	26.86	6.79	40.96	47.75	47.75
13-Jan-23	36.00	9.68	20.96	26.88	6.86	40.96	47.82	47.82
14-Jan-23	36.01	9.68	21.03	26.93	6.86	40.97	47.83	47.83
15-Jan-23	35.98	9.70	21.06	26.98	6.87	40.98	47.84	47.84
16-Jan-23	36.00	9.68	21.05	26.99	6.85	41.12	47.97	47.97
17-Jan-23	35.99	9.69	21.03	27.02	6.90	41.14	48.04	48.04
17-Jan-23	35.99	9.69	21.04	27.03	6.89	41.15	48.05	48.05
17-Jan-23	36.00	9.69	21.04	27.03	6.64	41.41	48.05	48.05
18-Jan-23	35.99	9.69	21.13	27.05	6.83	41.25	48.07	48.07
18-Jan-23	36.00	9.69	21.13	27.04	6.82	41.25	48.07	48.07
18-Jan-23	35.98	9.69	21.11	27.10	6.93	41.25	48.18	48.18
19-Jan-23	35.98	9.69	21.09	27.11	6.92	41.25	48.17	48.17
19-Jan-23	35.98	9.68	21.21	27.07	6.91	41.29	48.21	48.21
19-Jan-23	35.97	9.70	21.12	27.16	6.94	41.26	48.20	48.20
20-Jan-23	35.97	9.69	21.12	27.17	6.90	41.37	48.27	48.27
20-Jan-23	35.96	9.69	21.12	27.16	6.89	41.39	48.28	48.28
20-Jan-23	35.95	9.70	21.14	27.18	6.90	41.40	48.29	48.29
21-Jan-23	35.96	9.69	21.29	27.16	6.85	41.43	48.28	48.28
21-Jan-23	35.95	9.70	21.26	27.20	6.83	41.49	48.32	48.32
22-Jan-23	35.95	9.69	21.26	27.20	6.98	41.48	48.46	48.46
22-Jan-23	35.94	9.69	21.22	27.26	6.98	41.48	48.46	48.46
23-Jan-23	35.99	9.69	21.23	27.26	6.97	41.49	48.46	48.46
23-Jan-23	35.95	9.70	21.28	27.32	6.98	41.49	48.48	48.48
24-Jan-23	35.95	9.69	21.35	27.43	7.02	41.47	48.49	48.49
25-Jan-23	35.94	9.70	21.38	27.43	6.94	41.66	48.60	48.60
26-Jan-23	35.95	9.70	21.34	27.53	6.95	41.83	48.78	48.78
27-Jan-23	35.95	9.70	21.33	27.53	6.95	41.86	48.81	48.81
28-Jan-23	35.93	9.70	21.29	27.57	6.98	41.89	48.87	48.87
28-Jan-23	35.93	9.69	21.31	27.59	6.96	41.90	48.86	48.86
29-Jan-23	35.95	9.68	21.31	27.59	6.97	41.90	48.87	48.87
30-Jan-23	35.92	9.70	21.31	27.59	7.00	41.90	48.90	48.90

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
30-Jan-23	35.93	9.70	21.31	27.60	6.98	41.93	48.91	48.91
30-Jan-23	35.93	9.70	21.37	27.55	6.98	41.93	48.91	48.91
31-Jan-23	35.93	9.70	23.10	27.64	7.01	41.91	48.92	48.92
1-Feb-23	35.92	9.69	21.39	27.65	7.00	41.92	48.92	48.92
2-Feb-23	35.91	9.69	21.39	27.66	8.75	41.99	50.74	50.74
2-Feb-23	35.91	9.69	22.75	27.65	6.88	42.17	49.05	49.05
3-Feb-23	35.91	9.69	21.44	27.71	6.97	42.07	49.05	49.05
4-Feb-23	35.91	9.69	21.35	27.73	8.22	42.19	50.41	50.41
4-Feb-23	35.90	9.69	21.39	27.71	7.06	42.09	49.15	49.15
5-Feb-23	35.91	9.69	21.47	27.71	7.00	42.08	49.08	49.08
7-Feb-23	35.88	9.70	21.51	27.69	7.01	42.08	49.10	49.10
7-Feb-23	35.89	9.69	21.43	27.74	7.10	42.09	49.18	49.18
8-Feb-23	35.89	9.70	21.37	27.75	7.11	42.10	49.20	49.20
8-Feb-23	35.87	9.70	21.45	27.77	7.08	42.08	49.17	49.17
9-Feb-23	35.87	9.70	21.43	27.80	7.04	42.08	49.12	49.12
10-Feb-23	35.77	9.70	21.41	27.83	7.02	42.20	49.22	49.22
10-Feb-23	35.92	9.69	21.47	27.79	7.01	42.22	49.23	49.23
11-Feb-23	35.87	9.70	21.46	27.80	7.03	42.20	49.24	49.24
11-Feb-23	35.86	9.70	21.54	27.83	7.04	42.22	49.26	49.26
12-Feb-23	35.89	9.69	21.50	27.85	7.04	42.22	49.26	49.26
12-Feb-23	35.99	9.68	21.58	27.78	7.09	42.28	49.37	49.37
13-Feb-23	35.87	9.69	21.58	27.78	7.06	42.30	49.36	49.36
13-Feb-23	35.85	9.69	21.53	27.84	7.05	42.30	49.36	49.36
15-Feb-23	35.85	9.69	21.42	27.86	7.05	42.31	49.36	49.36
15-Feb-23	35.84	9.69	21.45	27.93	7.06	42.32	49.38	49.38
16-Feb-23	35.85	9.68	21.52	27.86	6.98	42.30	49.28	49.28
16-Feb-23	35.84	9.69	21.53	27.86	7.07	42.31	49.38	49.38
17-Feb-23	35.83	9.69	21.51	27.86	7.07	42.31	49.38	49.38
17-Feb-23	35.83	9.69	21.59	27.91	7.09	42.30	49.39	49.39
18-Feb-23	35.82	9.69	21.59	27.90	7.06	42.32	49.37	49.37
18-Feb-23	35.82	9.69	21.59	27.91	7.18	42.32	49.49	49.49
19-Feb-23	35.82	9.69	21.58	27.91	7.22	42.27	49.48	49.48
19-Feb-23	35.69	9.62	21.59	27.91	7.06	42.43	49.49	49.49
19-Feb-23	35.70	9.66	21.68	27.88	7.07	42.43	49.49	49.49
20-Feb-23	35.74	9.62	21.59	27.96	7.06	42.43	49.50	49.50
20-Feb-23	35.74	9.65	21.57	27.98	7.09	42.47	49.55	49.55

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
21-Feb-23	35.66	9.65	21.63	27.90	7.11	42.45	49.55	49.55
21-Feb-23	35.65	9.64	21.63	27.90	7.08	42.46	49.55	49.55
21-Feb-23	35.65	9.65	21.58	27.93	7.08	42.45	49.53	49.53
22-Feb-23	35.66	9.63	21.61	27.92	7.08	42.45	49.53	49.53
22-Feb-23	35.65	9.63	21.58	27.95	7.07	42.44	49.51	49.51
22-Feb-23	35.65	9.64	21.52	27.95	7.09	42.44	49.53	49.53
23-Feb-23	35.68	9.65	21.63	27.88	7.08	42.45	49.52	49.52
23-Feb-23	35.63	9.65	21.51	27.94	7.09	42.38	49.47	49.47
23-Feb-23	35.62	9.66	21.56	27.94	7.09	42.42	49.50	49.50
24-Feb-23	35.62	9.65	21.57	27.92	7.04	42.41	49.46	49.46
24-Feb-23	35.61	9.66	21.83	27.94	7.08	42.42	49.50	49.50
24-Feb-23	35.61	9.66	21.67	27.92	7.08	42.41	49.48	49.48
25-Feb-23	35.65	9.62	21.62	27.92	7.35	42.41	49.76	49.76
25-Feb-23	35.61	9.66	21.66	27.92	7.18	42.40	49.59	49.59
25-Feb-23	35.59	9.65	21.71	27.88	7.19	42.36	49.54	49.54
26-Feb-23	35.59	9.64	21.68	27.90	7.18	42.40	49.58	49.58
26-Feb-23	35.58	9.66	21.66	27.92	7.17	42.41	49.58	49.58
26-Feb-23	35.58	9.65	21.65	27.93	7.17	42.41	49.58	49.58
27-Feb-23	35.57	9.65	21.62	27.96	7.29	42.29	49.58	49.58
27-Feb-23	35.58	9.64	21.65	27.95	7.16	42.42	49.58	49.58
28-Feb-23	35.58	9.61	21.65	27.93	7.18	42.40	49.58	49.58
28-Feb-23	35.57	9.61	21.65	27.93	7.18	42.42	49.60	49.60
28-Feb-23	35.57	9.62			7.17	42.41	49.58	49.58

Appendix L Deformation Data from multipoint extensometer at Ch 0+67 m

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
6-Dec-22								
6-Dec-22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7-Dec-22	1.31	2.18	0.92	2.57	0.49	3.01	3.49	3.49
8-Dec-22	6.43	2.60	4.88	4.15	1.57	7.46	9.03	9.03
8-Dec-22	6.77	2.67	5.15	4.29	1.69	7.75	9.44	9.44
9-Dec-22	6.98	2.70	5.34	4.35	1.79	7.90	9.68	9.68
9-Dec-22	7.19	2.73	5.45	4.47	1.76	8.16	9.92	9.92
10-Dec-22	7.26	2.75	5.60	4.42	1.81	8.20	10.01	10.01
11-Dec-22	7.26	2.75	5.60	4.42	1.81	8.20	10.01	10.01
11-Dec-22	7.55	2.78	5.73	4.61	1.91	8.43	10.33	10.33
11-Dec-22	7.63	2.80	5.79	4.64	1.92	8.51	10.43	10.43
12-Dec-22	7.69	2.80	5.83	4.66	1.91	8.58	10.49	10.49
13-Dec-22	7.70	2.85	5.86	4.69	1.91	8.64	10.55	10.55
15-Dec-22	7.88	2.87	5.96	4.79	1.86	8.89	10.75	10.75
17-Dec-22	7.94	2.89	5.99	4.84	1.82	9.00	10.83	10.83
17-Dec-22	7.94	2.90	5.97	4.87	1.81	9.04	10.85	10.85
18-Dec-22	7.95	2.90	5.96	4.88	1.80	9.04	10.85	10.85
19-Dec-22	8.00	2.90	6.00	4.90	1.80	9.10	10.90	10.90
19-Dec-22	7.99	2.91	5.94	4.96	1.86	9.04	10.90	10.90
19-Dec-22	7.95	2.98	5.97	4.96	1.81	9.13	10.94	10.94
19-Dec-22	7.99	2.94	6.02	4.90	1.76	9.17	10.93	10.93
20-Dec-22	8.02	2.92	6.01	4.93	1.78	9.16	10.94	10.94
20-Dec-22	8.04	2.93	6.00	4.96	1.78	9.18	10.96	10.96
21-Dec-22	8.02	2.96	6.05	4.92	1.77	9.21	10.97	10.97
21-Dec-22	8.04	2.94	6.06	4.93	1.77	9.22	10.99	10.99
22-Dec-22	8.08	2.93	6.05	4.97	1.76	9.26	11.02	11.02
22-Dec-22	8.08	2.92	6.02	4.98	1.73	9.27	11.00	11.00
23-Dec-22	8.15	2.96	6.11	5.01	1.76	9.36	11.12	11.12
23-Dec-22	8.15	2.98	6.12	5.01	1.78	9.35	11.13	11.13
24-Dec-22	8.17	2.98	6.16	4.99	1.79	9.35	11.14	11.14
24-Dec-22	8.19	2.97	6.14	5.01	1.79	9.37	11.15	11.15
24-Dec-22	8.20	2.96	6.16	5.00	1.79	9.38	11.17	11.17

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
24-Dec-22	8.22	2.97	6.18	5.01	1.79	9.40	11.19	11.19
25-Dec-22	8.64	2.97	6.56	5.05	2.12	9.49	11.61	11.61
25-Dec-22	8.45	2.98	6.39	5.04	1.91	9.53	11.44	11.44
25-Dec-22	8.40	2.98	6.32	5.06	1.85	9.54	11.39	11.39
25-Dec-22	9.02	3.00	6.89	5.12	2.12	9.89	12.01	12.01
26-Dec-22	9.35	3.03	7.20	5.17	2.30	10.08	12.38	12.38
26-Dec-22	9.36	3.03	7.19	5.19	2.29	10.10	12.39	12.39
27-Dec-22	10.21	3.06	8.00	5.26	2.73	10.53	13.26	13.26
28-Dec-22	11.97	3.18	9.61	5.54	3.47	11.69	15.15	15.15
28-Dec-22	12.19	3.23	9.77	5.65	3.56	11.86	15.42	15.42
28-Dec-22	12.24	3.23	9.81	5.66	3.61	11.86	15.47	15.47
29-Dec-22	12.47	3.25	10.00	5.71	3.71	12.01	15.72	15.72
29-Dec-22	12.75	3.29	10.26	5.78	3.81	12.23	16.04	16.04
30-Dec-22	12.94	3.34	10.40	5.87	3.86	12.42	16.28	16.28
30-Dec-22	13.05	3.33	10.51	5.87	3.91	12.48	16.38	16.38
31-Dec-22	13.86	3.32	11.17	6.01	4.09	13.09	17.18	17.18
31-Dec-22	13.78	3.34	11.11	6.01	4.06	13.06	17.12	17.12
31-Dec-22	13.91	3.34	11.21	6.05	4.07	13.19	17.25	17.25
1-Jan-23	14.00	3.35	11.29	6.06	4.17	13.17	17.35	17.35
1-Jan-23	14.44	3.37	11.66	6.15	4.27	13.54	17.81	17.81
1-Jan-23	14.55	3.38	11.72	6.21	4.30	13.63	17.93	17.93
1-Jan-23	14.66	3.38	11.83	6.21	4.33	13.71	18.03	18.03
2-Jan-23	14.84	3.40	11.93	6.31	4.39	13.85	18.24	18.24
2-Jan-23	14.80	3.47	11.99	6.29	4.40	13.88	18.28	18.28
2-Jan-23	14.89	3.47	12.04	6.33	4.42	13.94	18.36	18.36
3-Jan-23	14.98	3.42	12.05	6.35	4.43	13.97	18.40	18.40
3-Jan-23	15.04	3.43	12.12	6.35	4.44	14.04	18.47	18.47
3-Jan-23	15.09	3.44	12.13	6.39	4.39	14.13	18.52	18.52
3-Jan-23	15.07	3.48	12.19	6.36	4.45	14.10	18.55	18.55
4-Jan-23	15.13	3.46	12.20	6.38	4.47	14.12	18.58	18.58
4-Jan-23	15.18	3.44	12.24	6.38	4.49	14.13	18.62	18.62
4-Jan-23	15.20	3.46	12.26	6.41	4.51	14.16	18.66	18.66
4-Jan-23	16.22	3.55	12.94	6.82	4.67	15.09	19.77	19.77

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
5-Jan-23	16.30	3.56	12.99	6.88	4.67	15.19	19.86	19.86
5-Jan-23	16.44	3.58	13.10	6.92	4.66	15.37	20.02	20.02
6-Jan-23	16.52	3.60	13.23	6.89	4.71	15.41	20.12	20.12
6-Jan-23	16.63	3.62	13.25	7.00	4.77	15.48	20.25	20.25
6-Jan-23	16.55	3.66	13.18	7.02	4.77	15.43	20.20	20.20
7-Jan-23	16.70	3.62	13.29	7.02	4.76	15.56	20.32	20.32
7-Jan-23	16.73	3.63	13.33	7.03	4.76	15.60	20.36	20.36
7-Jan-23	16.77	3.62	13.32	7.08	4.79	15.61	20.40	20.40
7-Jan-23	16.80	3.63	13.34	7.09	4.82	15.61	20.43	20.43
8-Jan-23	16.82	3.62	13.35	7.09	4.81	15.63	20.44	20.44
8-Jan-23	16.84	3.64	13.33	7.14	4.82	15.65	20.47	20.47
8-Jan-23	19.72	3.78	15.88	7.62	5.63	17.86	23.50	23.50
8-Jan-23	19.94	3.83	16.07	7.70	5.71	18.06	23.77	23.77
8-Jan-23	20.19	3.86	16.30	7.75	5.78	18.26	24.04	24.04
8-Jan-23	20.30	3.86	16.38	7.79	5.82	18.35	24.16	24.16
9-Jan-23	23.63	4.14	19.28	8.48	6.78	20.99	27.77	27.77
9-Jan-23	23.81	4.17	19.41	8.57	6.84	21.14	27.98	27.98
10-Jan-23	23.81	4.17	19.41	8.57	6.84	21.14	27.98	27.98
10-Jan-23	24.25	4.22	19.77	8.70	6.95	21.52	28.47	28.47
10-Jan-23	24.30	4.21	19.87	8.65	6.97	21.55	28.52	28.52
10-Jan-23	24.10	5.92	21.15	8.87	7.30	22.72	30.02	30.02
10-Jan-23	25.95	4.22	21.18	9.00	7.28	22.90	30.17	30.17
11-Jan-23	27.63	4.34	22.65	9.32	7.92	24.06	31.98	31.98
12-Jan-23	28.11	4.39	23.04	9.46	8.07	24.43	32.50	32.50
12-Jan-23	29.60	4.47	24.27	9.80	8.52	25.55	34.07	34.07
12-Jan-23	30.07	4.48	24.70	9.85	8.71	25.85	34.56	34.56
13-Jan-23	30.25	4.52	24.86	9.92	8.68	26.10	34.77	34.77
13-Jan-23	31.11	4.53	25.48	10.16	8.96	26.68	35.64	35.64
14-Jan-23	32.38	4.61	26.52	10.46	9.33	27.66	36.98	36.98
15-Jan-23	32.99	4.65	27.06	10.58	9.69	27.95	37.64	37.64
16-Jan-23	34.19	4.81	27.90	11.11	9.48	29.52	39.00	39.00
17-Jan-23	34.97	4.84	28.61	11.20	9.97	29.84	39.81	39.81
17-Jan-23	35.08	4.85	28.67	11.26	9.94	29.98	39.93	39.93

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
17-Jan-23	35.30	4.88	28.78	11.40	9.63	30.55	40.18	40.18
18-Jan-23	35.18	4.89	28.62	11.44	9.62	30.45	40.06	40.06
18-Jan-23	37.03	5.00	30.00	12.03	10.16	31.87	42.03	42.03
18-Jan-23	37.03	5.01	29.95	12.09	9.98	32.06	42.03	42.03
19-Jan-23	36.95	5.08	30.00	12.03	10.16	31.87	42.03	42.03
19-Jan-23	37.51	5.04	30.30	12.26	10.17	32.39	42.56	42.56
19-Jan-23	38.69	5.07	31.16	12.61	10.48	33.28	43.76	43.76
20-Jan-23	38.88	5.08	31.26	12.69	10.44	33.52	43.96	43.96
20-Jan-23	38.96	5.09	31.27	12.78	10.38	33.67	44.05	44.05
20-Jan-23	39.53	5.15	31.89	12.78	10.49	34.18	44.68	44.68
21-Jan-23	39.78	5.13	31.78	13.13	10.51	34.40	44.92	44.92
21-Jan-23	39.86	5.08	31.79	13.16	10.49	34.45	44.94	44.94
22-Jan-23	40.25	5.09	32.00	13.34	10.54	34.80	45.34	45.34
23-Jan-23	40.46	5.07	32.11	13.42	10.56	34.97	45.53	45.53
23-Jan-23	40.70	5.09	32.27	13.53	10.57	35.23	45.80	45.80
24-Jan-23	40.78	5.14	32.32	13.60	10.58	35.34	45.92	45.92
25-Jan-23	41.14	5.09	32.42	13.80	10.62	35.60	46.22	46.22
26-Jan-23	41.14	5.10	32.45	13.79	10.66	35.58	46.24	46.24
27-Jan-23	41.23	5.12	32.51	13.84	10.63	35.73	46.35	46.35
28-Jan-23	41.04	5.13	32.26	13.92	10.32	35.86	46.18	46.18
28-Jan-23	41.36	5.14	32.58	13.92	10.62	35.88	46.50	46.50
29-Jan-23	41.35	5.16	32.76	13.75	10.63	35.88	46.51	46.51
30-Jan-23	41.34	5.17	32.53	13.98	9.08	37.43	46.51	46.51
30-Jan-23	41.36	5.18	32.55	13.99	9.10	37.44	46.54	46.54
30-Jan-23	41.51	5.17	32.65	14.03	10.65	36.02	46.68	46.68
31-Jan-23	41.53	5.23	32.68	14.09	10.65	36.11	46.76	46.76
1-Feb-23	41.56	5.21	32.63	14.14	10.66	36.11	46.77	46.77
2-Feb-23	41.57	5.21	32.68	14.09	10.65	36.13	46.78	46.78
2-Feb-23	41.57	5.21	32.65	14.12	10.60	36.17	46.78	46.78
3-Feb-23	41.63	5.21	32.74	14.11	10.66	36.19	46.85	46.85
4-Feb-23	41.28	5.33	32.68	13.93	10.32	36.28	46.61	46.61
4-Feb-23	41.70	5.23	32.74	14.18	10.66	36.26	46.93	46.93
5-Feb-23	41.68	5.24	32.73	14.19	10.63	36.28	46.92	46.92

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
6-Feb-23	41.64	5.27	32.71	14.19	10.62	36.28	46.90	46.90
7-Feb-23	41.89	5.22	32.86	14.25	10.78	36.32	47.10	47.10
7-Feb-23	41.86	5.22	32.84	14.23	10.75	36.33	47.08	47.08
8-Feb-23	41.83	5.23	32.65	14.41	10.68	36.38	47.06	47.06
8-Feb-23	41.85	5.23	32.80	14.29	10.66	36.42	47.08	47.08
9-Feb-23	41.86	5.25	32.82	14.28	10.62	36.49	47.10	47.10
10-Feb-23	41.86	5.25	32.81	14.29	10.68	36.43	47.11	47.11
10-Feb-23	41.86	5.25	32.81	14.29	10.66	36.45	47.10	47.10
11-Feb-23	41.87	5.25	32.81	14.30	10.65	36.47	47.11	47.11
11-Feb-23	41.90	5.25	32.83	14.32	10.68	36.47	47.15	47.15
12-Feb-23	41.90	5.25	32.84	14.32	10.68	36.47	47.16	47.16
12-Feb-23	41.93	5.24	32.86	14.32	10.64	36.54	47.17	47.17
13-Feb-23	41.95	5.26	32.84	14.37	10.68	36.53	47.21	47.21
13-Feb-23	41.96	5.27	32.77	14.45	10.69	36.54	47.23	47.23
15-Feb-23	41.97	5.27	32.86	14.38	10.68	36.56	47.24	47.24
15-Feb-23	41.97	5.27	32.86	14.37	10.69	36.54	47.23	47.23
16-Feb-23	42.01	5.25	32.87	14.39	10.69	36.57	47.26	47.26
16-Feb-23	42.04	5.26	32.91	14.39	10.80	36.50	47.30	47.30
17-Feb-23	42.07	5.21	32.87	14.41	10.68	36.60	47.28	47.28
17-Feb-23	42.31	5.27	33.17	14.41	10.98	36.59	47.58	47.58
18-Feb-23	42.30	5.27	33.16	14.41	10.96	36.61	47.57	47.57
18-Feb-23	42.31	5.27	33.17	14.41	10.98	36.59	47.58	47.58
19-Feb-23	42.06	5.27	32.89	14.44	10.69	36.63	47.33	47.33
19-Feb-23	42.05	5.28	32.90	14.43	10.70	36.63	47.33	47.33
21-Feb-23	42.01	5.30	32.86	14.45	10.62	36.68	47.31	47.31
21-Feb-23	42.06	5.31	32.96	14.42	10.69	36.69	47.37	47.37
21-Feb-23	42.07	5.30	32.86	14.52	10.70	36.68	47.38	47.38
22-Feb-23	42.07	5.28	32.88	14.47	10.52	36.83	47.35	47.35
22-Feb-23	42.09	5.28	32.92	14.46	10.70	36.68	47.38	47.38
22-Feb-23	42.09	5.28	32.90	14.47	10.69	36.68	47.37	47.37
23-Feb-23	42.17	5.28	32.96	14.49	10.77	36.67	47.45	47.45
23-Feb-23	42.18	5.26	32.98	14.46	10.76	36.68	47.44	47.44
23-Feb-23	42.16	5.27	32.95	14.48	10.72	36.71	47.43	47.43

Depth of Anchor	2 m		5 m		10 m		17 m	
Observation Date	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)	Relative Displacement (mm)	absolute Displacement (mm)
24-Feb-23	42.14	5.24	32.85	14.52	10.65	36.73	47.37	47.37
24-Feb-23	42.09	5.26	32.94	14.41	10.71	36.64	47.35	47.35
24-Feb-23	42.09	5.26	32.98	14.37	10.72	36.64	47.35	47.35
25-Feb-23	42.10	5.26	32.90	14.46	10.69	36.67	47.36	47.36
25-Feb-23	42.09	5.26	32.89	14.47	10.68	36.67	47.35	47.35
25-Feb-23	42.14	5.26	32.93	14.47	10.72	36.67	47.40	47.40
26-Feb-23	42.10	5.26	32.90	14.46	10.69	36.67	47.36	47.36
26-Feb-23	42.13	5.26	32.92	14.47	10.70	36.69	47.39	47.39
26-Feb-23	42.13	5.26	32.91	14.48	10.56	36.83	47.39	47.39
27-Feb-23	42.12	5.26	32.90	14.48	10.56	36.83	47.38	47.38
27-Feb-23	42.14	5.26	32.92	14.48	10.71	36.70	47.41	47.41
28-Feb-23	42.14	5.26	32.91	14.48	10.69	36.71	47.40	47.40
28-Feb-23	42.14	5.27	32.92	14.49	10.69	36.72	47.41	47.41
28-Feb-23	42.15	5.27	32.91	14.51	10.68	36.73	47.42	47.42

Appendix M Powerhouse convergence data at Ch 0+63 m

Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2022-11-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7484	0.0058	935.3111
2022-11-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7480	0.0050	935.3098
2022-11-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7483	0.0052	935.3099
2022-11-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7485	0.0055	935.3103
2022-11-16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7486	-0.0054	935.3105
2022-11-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7660	-0.0052	935.3127
2022-11-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7480	-0.0053	935.3104
2022-11-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7465	-0.0055	935.3120
2022-11-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7449	-0.0130	935.3142
2022-11-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7452	-0.0132	935.3140
2022-11-26	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7653	-0.0096	935.3134
2022-11-27	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7655	-0.0099	935.3132
2022-11-28	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7465	-0.0055	935.3122
2022-11-29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7449	-0.0130	935.3144
2022-11-30	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7458	-0.0123	935.3132

Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2022-12-01	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7460	-0.0125	935.3134
2022-12-02	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7485	0.0055	935.3103
2022-12-03	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7490	-0.0059	935.3105
2022-12-04	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7536	0.0048	935.3278
2022-12-05	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7555	0.0031	935.3210
2022-12-06	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7449	-0.0130	935.3144
2022-12-07	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7458	-0.0123	935.3132
2022-12-08	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7480	0.0075	935.3300
2022-12-09	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7496	-0.0049	935.3515
2022-12-10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7661	0.0177	935.3670
2022-12-11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7658	0.0153	935.3799
2022-12-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7651	-0.0142	935.3937
2022-12-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7672	0.0174	935.4039
2022-12-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7678	0.0168	935.4034
2022-12-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7632	0.0278	935.3972
2022-12-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7621	0.0168	935.3952
2022-12-18	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7640	0.0056	935.3959
2022-12-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7645	0.0061	935.3952
2022-12-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7640	0.0076	935.3949
2022-12-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7653	0.0081	935.3941
2022-12-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7586	0.0215	935.3787
2022-12-23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7580	0.0226	935.3781
2022-12-24	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7590	0.0227	935.3785
2022-12-25	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7636	0.0305	935.3747
2022-12-26	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7642	0.0313	935.3744
2022-12-27	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7609	0.0363	935.3661
2022-12-28	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7615	0.0342	935.3672
2022-12-29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7630	0.0349	935.3674
2022-12-30	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7656	0.0339	935.3671
2022-12-31	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7687	0.0319	935.3683
2023-01-01	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7609	0.0342	935.3677
2023-01-02	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7610	0.0356	935.3670
2023-01-03	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7620	0.0362	935.3660
2023-01-04	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7592	0.0345	935.3684
2023-01-05	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7590	0.0346	935.3682
2023-01-06	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7602	0.0264	935.3680
2023-01-07	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7606	0.0266	935.3580
2023-01-08	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7603	0.0267	935.3578

Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2023-01-09	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7599	0.0264	935.3569
2023-01-10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7580	0.0255	935.3666
2023-01-11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7618	0.0268	935.3651
2023-01-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7585	0.0224	935.3656
2023-01-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7599	0.0264	935.3569
2023-01-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7585	0.0424	935.3656
2023-01-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7602	0.0501	935.3657
2023-01-16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7601	0.0519	935.3642
2023-01-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7606	0.0529	935.3642
2023-01-18	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7585	0.0424	935.3656
2023-01-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7598	0.0321	935.3636
2023-01-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7623	0.0313	935.3627
2023-01-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7661	0.0252	935.3641
2023-01-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7615	0.0561	935.3627
2023-01-23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7641	0.0594	935.3635
2023-01-24	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	39.7599	0.0421	935.3636

Appendix N Powerhouse convergence data at Ch 0+63 m

Powerhouse: Ch. 0+63 m									
All displacements shown in scale [mm]; tunnel contours not in scale									
Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2022-12-09	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4131	-0.0044	935.3332
2022-12-10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4135	-0.0054	935.3318
2022-12-11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4130	-0.0220	935.3315
2022-12-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4132	-0.0247	935.3283
2022-12-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4145	-0.0269	935.3295
2022-12-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4138	-0.0257	935.3299
2022-12-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4123	-0.0098	935.3283
2022-12-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4145	-0.0277	935.3277
2022-12-18	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4130	-0.0428	935.3279
2022-12-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4102	-0.0351	935.3277
2022-12-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4105	-0.0356	935.3267
2022-12-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4110	-0.0349	935.3272
2022-12-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4102	-0.0351	935.3277

Powerhouse: Ch. 0+63 m									
All displacements shown in scale [mm]; tunnel contours not in scale									
Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2022-12-23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4149	-0.0017	935.3256
2022-12-24	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4156	-0.0044	935.3211
2022-12-25	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4189	-0.0061	935.3176
2022-12-26	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4185	-0.0057	935.3173
2022-12-27	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4156	0.0344	935.3211
2022-12-28	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4160	0.0351	935.3209
2022-12-29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4172	0.0343	935.3220
2022-12-30	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4156	0.0332	935.3235
2022-12-31	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4156	0.0355	935.3215
2023-01-01	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4169	0.0331	935.3206
2023-01-02	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4153	0.0320	935.3201
2023-01-03	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4144	0.0309	935.3200
2023-01-04	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4135	0.0309	935.3210
2023-01-05	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4126	0.0309	935.3260
2023-01-06	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4117	0.0309	935.3280
2023-01-07	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4108	0.0309	935.3320
2023-01-08	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4099	0.0309	935.3355
2023-01-09	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4090	0.0309	935.3390
2023-01-10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4081	0.0309	935.3425
2023-01-11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4080	0.0309	935.3460
2023-01-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4079	0.0309	935.3495
2023-01-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4078	0.0309	935.3530
2023-01-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4077	0.0309	935.3565
2023-01-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4076	0.0309	935.3600
2023-01-16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4075	0.0309	935.3635
2023-01-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4074	0.0309	935.3670
2023-01-18	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4073	0.0309	935.3705
2023-01-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4072	0.0309	935.3740
2023-01-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4071	0.0309	935.3775
2023-01-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3810
2023-01-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3845
2023-01-23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-24	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880

Powerhouse: Ch. 0+63 m									
All displacements shown in scale [mm]; tunnel contours not in scale									
Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2023-01-25	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-26	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-27	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-28	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-29	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-30	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-01-31	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-01	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-02	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-03	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-04	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-05	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-06	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-07	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-08	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-09	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-11	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-12	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-13	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-14	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-15	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-16	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	59.4070	0.0309	935.3880
2023-02-17	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-18	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-19	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-20	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-21	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-22	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-23	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-24	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-25	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-26	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880

Powerhouse: Ch. 0+63 m

All displacements shown in scale [mm]; tunnel contours not in scale

Monitoring Period	Right Crown			Left			Center Crown		
	X	Y	H	X	Y	H	X	Y	H
2023-02-27	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880
2023-02-28	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000		0.0309	935.3880